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U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

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U.S. DEPARTMENT OF COMMERCE
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A Study of 34-E Dual-Drum Pavers

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by DONALD O. WOOLF,
Highway Physical Research Engineer

This article summarizes data obtained in 16 studies of the mixing of portland cement concrete in 34-E dual-drum pavers, conducted by 13 State highway departments. Determinations were made of the desirable time of mixing and the amount of overload which could be permitted. The results indicated that the highest strengths were attained at a 60-second mixing time, exclusive of transfer time, and that an overload of 10 or 20 percent could be tolerated. It was also found that very little loss in strength of the concrete resulted when the mixing time was reduced to 40 seconds, exclusive of transfer time, but when the mixing time was reduced to 30 seconds a loss in strength of 5 to 6 percent occurred. A mixing time of more than 60 seconds, exclusive of transfer time, was found to be undesirable since it involved waste of effort and resulted in loss of strength without gain in the uniformity of the concrete.

IN December 1957, the Bureau of Public Roads suggested that a study of the production of portland cement concrete in 34-E dual-drum pavers be made by interested State highway departments, to determine the most desirable mixing time and the permissible overload, if any. State highway specifications showed that, exclusive of the transfer time required to move the concrete from the first to the second drum, some States permitted a mixing time of as little as 50 seconds, while other States required mixing times of as much as up to 120 seconds. Some State specifications permitted no overload, but others allowed overloads of from 10 to 20 percent. It was believed that factual information on the effect of mixing time and of amount of overload might permit the more rapid production of concrete without reduction in quality. Lower cost of the concrete in place should then be obtained.

The program proposed by the Bureau of Public Roads suggested that mixing times of 30, 60, and 90 seconds, exclusive of transfer time, be used. Since most States used a mixing time of about 60 seconds, this was taken as a standard of comparison. The 30-second time was suggested as part of the program to provide concrete which might possibly be undermixed; the 90-second time was intended to determine if extended mixing of concrete would abrade the aggregate to such an extent that excessive fines would be produced, causing the strength of the concrete to suffer. The States were informed, however, that these times could be changed to suit their particular interests, if they so desired.

As proposed in the program, many States used an overload of 10 percent in their studies, and in some cases an overload of 20 percent was used. Where the project contractors objected to the larger overload because of possible resultant wear on their equipment and waste of material, the 20-percent overload was omitted.

The suggested program also recommended that for a given combination of mixing time and overload, samples for test be taken from each tenth batch of 90 consecutive batches of concrete. It was further recommended that three samples be taken from each batch sampled, one sample representing each third of the concrete as discharged from the bucket. Specimens for compression and flexure tests were made from certain of the batches sampled and tests for air content, unit weight, and uniformity of composition were made on certain other batches.

Conclusions

The data obtained by the States were so extensive that only a considerably condensed review can be included in this report. Based on a painstaking study of all material reported, the following conclusions have been drawn.

The results of strength tests included in this investigation showed that for 34-E dual-drum pavers, the greatest strength was obtained with a mixing time of 60 seconds, not including transfer time between drums. This mixing time could be reduced to 40 seconds, exclusive of transfer time, with but very little reduction in the strength of the concrete. Reduction of the mixing time to 30 seconds, exclusive of transfer time, caused a reduction in strength of 5 to 6 percent. Concrete mixed for 30 seconds occasionally showed segregation but generally could be placed and finished without difficulty. In some studies it was found desirable to increase the water-cement ratio slightly for concrete mixed for only 30 seconds.

A mixing time of more than 60 seconds, exclusive of transfer time, was found to be undesirable, since it involved waste of effort and resulted in loss of strength without gain in the uniformity of the concrete. Little evidence of excessive abrasion of the aggregate was noted, however, even with mixing times as long as 90 seconds.

Overloading of 34-E dual-drum pavers above their rated capacity caused a reduction in strength of 1 to 2 percent for an overload of 20 percent, and a reduction of 2 to 4 percent for an overload of 10 percent. The greater reduction for the smaller overload may have been fallacious, or it may have been possible that the mixers used in these tests actually performed better with a greater load. Data to clarify this point were not available.

In studies of the uniformity of concrete produced by 34-E dual-drum pavers, it was found that the quality of concrete depended on the operations preceding the paver mixing operation. If a harsh mix was fed to the paver, extended mixing time still furnished a harsh concrete. If a properly sized and proportioned mix was used, most, if not all, of the mixers studied in this investigation furnished well mixed concrete after only 30 seconds of mixing.

It should be noted that some contractors objected to an overload of more than 10 percent on the grounds that their equipment would not handle it. Also, some contractors were not able to use a mixing time shorter than 50 seconds, due to their inability to supply materials to the paver.

In general, it is believed that the results of the strength tests of concrete were more indicative of the effectiveness of the mixers used than were the results of tests for uniformity of concrete. The tests made for the latter purpose were of considerable value, but if studies of this type should be conducted in the future, a marked increase in the number of tests for compressive strength would be recommended, with elimination of the flexural strength tests and a reduction in the tests for uniformity. If it could be devised, a test for cement content in the mixed concrete which could be made in entirety on the project would be recommended, with another test for water content of the concrete.

Scope of Data

The studies of the effect of mixing time and overload on the quality of portland cement concrete produced in 34-E dual-drum pavers were made by 13 State highway departments; 2 studies were made by each of 3 States, resulting in a total of 16 studies. The locations of the studies are shown in table 1. Information concerning the type of mixer used, the coarse aggregate, the nominal mixing times, and the overloads, is given in table 2.

In this article, data from the studies are presented on the effect of mixing time and overload on the compressive and flexural strengths of concrete. Also reported are determinations intended to show the uniformity (or lack of it) of single batches of concrete, as well as variations from batch to batch of concrete prepared under presumably the same

---

[1] This article was presented at the 39th Annual Meeting of the Highway Research Board, Washington, D.C., January 1960.

Table 1.—Highway construction projects in the study of 34-E dual-drum pavers

State	Project number	Date of study	Highway route number	Location
Alabama	I-65-3 (14) (15)	Oct. 1958	U.S. 31	25 miles north of Birmingham
California	III-Pla-17-A, Roe, B.	July 1959	U.S. 40	Near Roseville
Delaware	F-12(4) S-US-48(2).	July 1958	Del. 8	Dover, Division St. and Forrest Ave.
District of Columbia.	F-55(1)	June 1958		Irving St. NE.
Florida	I-4-1(2)	July 1958	U.S. 4	Near Plant City
Kansas	F-0 92	Oct. 1958	U.S. 36	Beattie Corner to Nemaha County line
Michigan	I-02-5(6)	June 1958	U.S. 12	Ypsilanti, near Willow Run Airport
Nebraska	F-250(7)	Oct. 1958	U.S. 77	Wymore to Kansas State line
New York 1	BT-57-1	Aug. 1958		Near East Chatham
New York 2	I-1119(11)	Oct. 1958		20 miles west of Albany
Ohio 1	I-196(5)	June 1958	U.S. 25	Wapakoneta bypass
Ohio 2	I-529(11)	Aug. 1958	U.S. 40	Kirkersville bypass
Virginia 1	F FG-018-1(1)	Sept. 1958	U.S. 29	Lynchburg bypass
Virginia 2	U-101-1(3)	Oct. 1958	U.S. 29	Arlington County, Lee Highway
Washington	Cont. 5733	Oct. 1958		Olympia
West Virginia	S-2(4)	Oct. 1958	W. Va. 14	Charleston to airport

Table 2.—Type of paver, aggregate, and principal variables used in the 16 study projects

State	Paver	Coarse aggregate		Principal variables	
		Type	Maximum size	Nominal mixing times used	Overload
			Inches	*Seconds*	*Percent*
Alabama	A	Stone	1½	30-50-90	0-10-20
California	B	Gravel	1½	30-50-70	0-10-20
Delaware	A	Stone	2	40-70-90	0-10-20
District of Columbia.	A	Gravel	2	30-50-70	0-10-20
Florida	B	Stone	2	30-60-90	0-10-20
Kansas	A	Sand-gravel	1	50-60-70-90	0-10-20
Michigan	A	Slag	2	30-50-70	0-10-20
Nebraska	C	Stone	1	30-50-60-90	0-10-20
New York 1	B	Stone	2	40-50-60-90-120	0-10-12-20
New York 2	B	Crushed gravel.	2	60-90-120	10-20
Ohio 1	B	Stone	1½	40-50-75	0-10-20
Ohio 2	A	Gravel	-------	50-75	0-10-20
Virginia 1	B	Stone	2½	30-60-90	0-10-20
Virginia 2	D	Gravel	2½	30-60-90	0-10
Washington	B	Gravel	-------	45-60	10-20
West Virginia	C	Gravel	2	30-40-50-60	0-10

mixing conditions. The types of tests made by the several States are indicated in table 3.

Some of the States prepared formal reports at the conclusion of their individual studies, while others supplied test results in an informal manner. The data obtained by all States were so extensive that only a review of them can be included in this report. Of particular interest, among data excluded here, were those that would permit comparing the air content of concrete as determined by the Chace air meter and standard pressure or volumetric meters, and comparisons between the results of slump cone and Kelly ball tests. It is expected that these data will be summarized and reported at a future time, but their inclusion in this report was not considered necessary.

The study was unusual in that it involved considerable effort and cooperation on the part of a number of State highway departments and highway contractors. The skillful handling of difficult field testing operations by the personnel involved, often under unfavorable conditions, contributed greatly to the success and value of the study.

Procedure Problems

In determinations of the mixing time, in this article, the transfer time between drums was not included. It has been stated by some that the concrete is mixed during this transfer process. However, in their reports different States reported different lengths of transfer time, even for the same make of paver, and it was believed that it would simplify the presentation of the material if only the mixing time in the drums was considered. The values reported by the States for mixing times are shown in table 4.

The original program proposal recommended sampling of every 10th batch for testing. Some States believed that sampling of every 10th batch might be too frequent, and sampled every 20th batch instead. Where the testing staff was limited, this allowed ample time for the test procedures. Some States questioned some of the methods of test proposed, and modified or eliminated them in accordance with their experience or needs. One State cast only a few specimens for strength tests, and drilled cores from the pavement for the strength tests of record. Other differences were found among the individual studies. In general, however, the recommended program was followed sufficiently to permit use of most of the data obtained.

One of the most controversial items in the suggested program was the method of obtaining the three samples of concrete representing a single batch or bucket load. It was recom-

Table 3.—Various tests on concrete performed by the States [1]

State	Tests for compressive strength on cylinders [2]			Tests for flexural strength [3]			Tests for uniformity of plastic concrete							
							Washout		Unit Weight	Air content [4]		Slump	Kelly ball	Willis-Hime
	7 days	14 days	28 days	7 days	14 days	28 days	Fine	Coarse		Pressure	Chace			
Alabama	X	----	X	X	----	X	X	----	X	X	X	X	X	----
California	----	X	----	----	----	----	X	X	----	X	----	----	X	----
Delaware	X	X	X	----	X	----	X	X	X	X	X	X	X	----
District of Columbia	X	X	X	----	X	----	X	X	X	X	X	X	X	X
Florida	X	X	X	X	X	X	----	----	X	X	X	X	X	----
Michigan	----	----	X	----	----	X	X	X	X	----	X	X	X	----
Nebraska	X	----	X	X	----	X	----	X	X	X	X	X	X	----
New York	----	----	----	----	----	----	----	X	X	X	X	X	----	X
Ohio 1	----	----	X	X	X	----	----	X	X	X	X	X	X	----
Ohio 2	----	----	X	X	X	----	----	X	X	X	X	X	X	----
Virginia 1	----	----	X	----	X	----	X	X	X	X	X	X	X	X
Virginia 2	X	X	X	X	----	X	X	X	X	X	X	X	X	X
Washington	----	----	X	----	----	X	----	----	X	----	----	X	X	----
West Virginia	X	X	X	X	X	----	X	X	X	X	X	X	X	----

[1] No information was received from Kansas. [2] Michigan and New York used cores to test compressive strength. [3] The District of Columbia conducted a 3-day test for flexural strength. [4] Alabama and Michigan used the rollermeter method for testing air content.

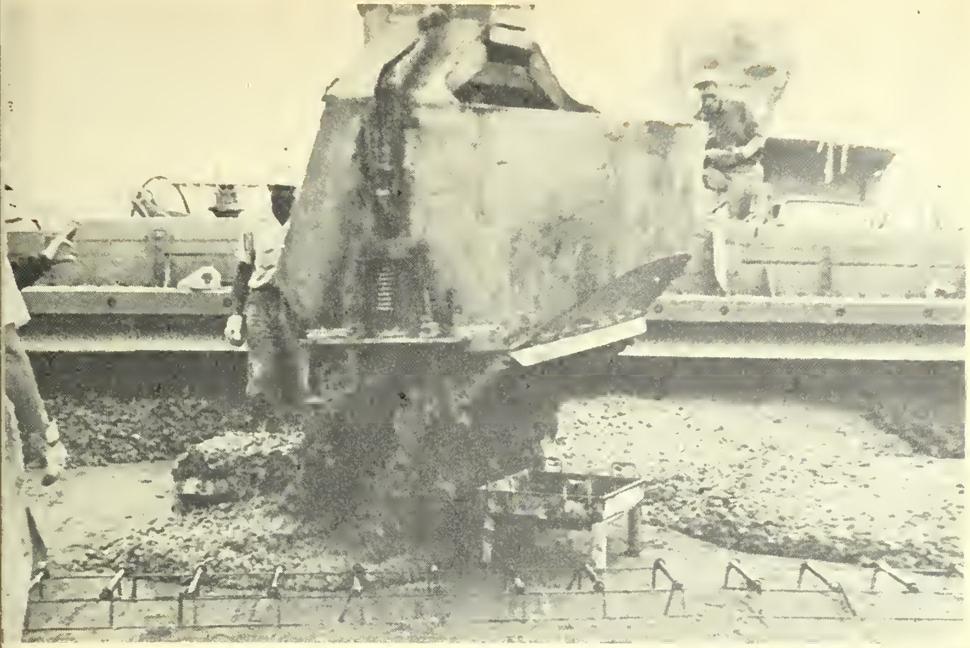


Figure 1.—One of the methods used in obtaining sample concrete.

ended originally that three pans be placed n the subgrade, slightly separated to cover e area over which a bucket load of concrete ould be distributed. However, some diffi-lty was found with this method of sampling. hen the pans were placed directly on the bgrade, the wave of concrete from the icket frequently knocked the pans aside. some projects, to correct this difficulty, e pans were held in place by concrete oveled against their sides. Another method und to be effective was mounting the pans legs about 12 inches high, as shown in ure 1.

Some States adopted the practice of having e bucket load dumped on the subgrade her in a ribbon or in an oval-shaped pile. Samples possibly representing different sections of the bucket-load were then taken from the concrete. At first it was questionable whether the samples so obtained could be considered to represent the first, middle, and last third of the concrete batch discharged from the mixer. However, it was pointed out that the first third of the concrete discharged from the bucket might not be the same as the first third of the concrete discharged from the mixer. Due to restraint of the sides of the bucket, the first third of concrete discharged from the bucket may include some of the second third of the concrete discharged into the bucket. For this reason, no consideration was given in this report to differences between the methods used to obtain these one-third-bucket-load samples. Although such differences existed, it may be considered that the collective samples for each mixing time and overload are reasonably representative of the entire mixing load.

Table 4.—Average mixing time, in seconds, as given by those States where mixing and transfer times were recorded [1]

State	Nominal mixing time	No overload		10 percent overload		20 percent overload		Average actual mixing time
		Transfer time	Actual mixing time	Transfer time	Actual mixing time	Transfer time	Actual mixing time	
Alabama	30	11	30	15	30	11	29	30
	50	11	49	11	49	11	49	49
	90	11	79	11	75	11	79	78
California	30	10	21	10	23	10	24	23
	50	10	40	10	39	10	38	39
	70	10	60	10	60	10	60	60
Delaware	40	9	41	9	41	9	41	41
	70	9	70	9	70	9	70	70
	90	9	90	9	90	9	90	90
District of Columbia	30	12	29	12	30	12	30	30
	50	12	45	12	45	12	50	47
	70	12	70	12	70	12	69	70
Kansas	50	10	39	10	54	10	51	48
	60	10	60	10	57	10	53	57
	70	10	60	10	62	10	61	61
	90	10	75	10	76	10	77	76
Ohio 1	40	8	41	9	43	8	42	42
	50	8	51	9	54	9	51	52
	75	9	66	9	66	9	65	66
Ohio 2	50	10	50	10	50	12	49	50
	75	10	65	10	65	12	64	65
Virginia 1	30	------	27	------	28	------	28	28
	60	------	58	------	60	------	60	59
	90	------	89	------	87	------	90	89
Virginia 2	30	------	29	------	36	------	------	32
	60	------	58	------	63	------	------	60
	90	------	93	------	96	------	------	94

[1] Florida (30, 60, 90 sec.), Michigan (30, 50, 70 sec.), Nebraska (30, 50, 60, 90 sec.), and West Virginia (30, 40, 50, 60 sec.) did not include transfer time in mixing times stated. New York included a transfer time of 9 sec. in mixing times of 40, 50, 60, 90, and 120 sec. No information of transfer time was furnished by Washington (45, 60 sec.).

Effect of Mixing Time on Strength

As shown in table 2, the States used widely different nominal mixing times, and some differences in overloads. There also was a variance in the materials used and in the methods of testing compressive and flexural strengths. To place each set of data on a uniform basis for comparison, the strength obtained for a 60-second mixing time, or with no overload, was adopted as the standard of comparison. Strengths obtained for other mixing times, or for overloads, were then compared with these values.

In six of the studies, a 60-second mixing time was not used. To permit comparison of the data obtained with those for the other projects, the determined strengths were plotted against mixing time and a curve drawn through the points. The value shown on this curve at intersection with the 60-second abscissa was taken as the base measure.

To keep the reporting of data concerning mixing time within reasonable bounds, some liberties have been taken in their presentation. The mixing times are shown at 5-second intervals, and a value for strength obtained with an actual mixing time of 48 seconds, for example, is shown for 50 seconds. However, if there was only one determination of strength of concrete for a given time of mixing, this was not shown in the charts. In many instances the elimination of these single values removed wild results and showed to better advantage the trend of the remaining mutually supporting data.

Data reported by each State of the effect of mixing time on the compressive and flexural strength of concrete are shown in figures 2 and 3. The varying strengths reflect the materials used and the methods of testing employed. Some States used cantilever beam testing machines, and obtained flexural strengths much higher than those using center-point loading.

Data averaged for all of the studies to show the effect of mixing time on the compressive strength are plotted in figure 4. The curve in each portion of the figure presents the best considered average. In the lowest portion of the figure, the curve represents an average of the other three curves. Similar curves for flexural strength are shown in figure 5.

The same finding; that the compressive and flexural strengths of concrete prepared with the mixers and materials used were at a maximum for a mixing time of 60 seconds, is evident in figures 4 and 5. Marked increase or decrease in mixing time resulted in a reduction in strength, but this reduction was of no great moment. For a mixing time of only 30 seconds, the compressive strength was reduced only 6 percent and the flexural strength only 5 percent. For a mixing time of 90 seconds, the compressive strength was reduced only 7 percent and the flexural strength only

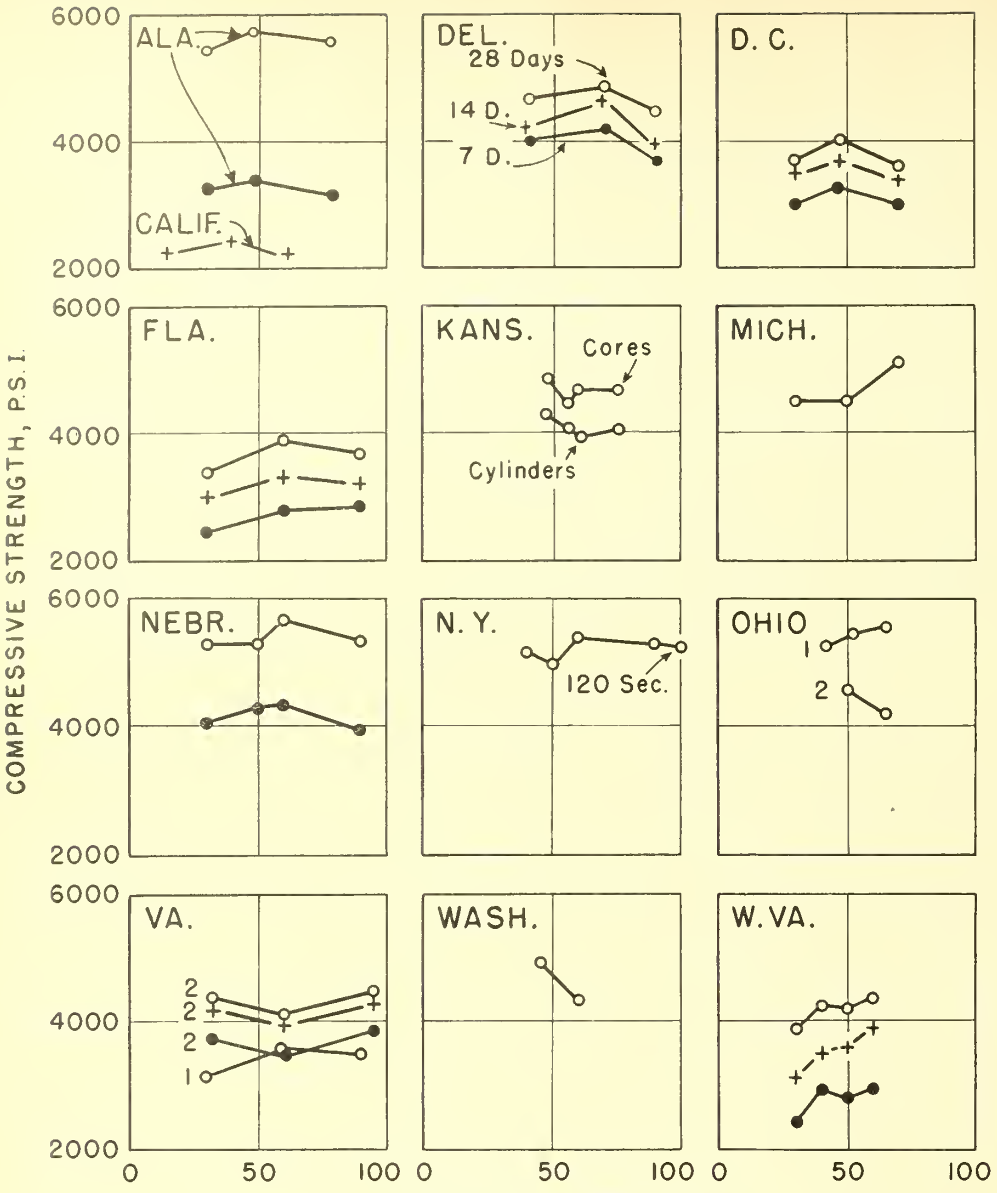


Figure 2.—Effect of mixing time on compressive strength, by State.

3 percent. A reduction in mixing time to 50 seconds caused a reduction in compressive strength of about 0.5 percent and no decrease in flexural strength.

Figures 6 and 7 present the effect of ove load on the compressive and flexural strengt of the samples tested. The curve representin the average for all determinations of con pressive strength shows that a decrease of percent in strength resulted for an overloa of 10 percent. For a 20-percent overload, decrease in strength of only 1 percent resulte In the flexural strength determinations, ove loads of 10 and 20 percent each caused reduction in strength of only 2 percent.

Some inquiry has been made of the reaso for the lower compressive strength of concret prepared with an overload of 10 percent tha with an overload of 20 percent. It wa speculated that the greater overload cause the drum to operate more efficiently; howeve if this were the reason a mixer with an ove load of 10 percent would be expected t operate more efficiently than one with r overload. Such was not found to be the cas if strength tests of concrete are used to judg the efficiency or rather the effectiveness of mixer. Some other reason must apply t these test results.

Most specimens were prepared, cured, an tested in accordance with standard AST methods. No concern need be given to th phase of each investigation, as the data we apparently affected but little by variations the preparation of specimens. With this a cepted, it may be assumed that variations the strength of the concrete are due to tl mixing procedures used, or the weather co ditions over which there was no control.

Further study was given to the effect overload on the compressive strength of co crete, reported in table 5. It was observe that in Florida the strength for no overlo was considerably higher than that for percent overload at all ages and for 20 perce overload at an age of 7 days. It was believe that a detailed study of the results from th State might be warranted. It was also not that in the Virginia No. 2 study and in tl West Virginia study the results for a 1 percent overload ranged from slightly lower considerably lower than for no overload, a that no tests were made for an overload of percent. It was believed that these resu

Table 5.—Effect of overload on compressive strength

State	Tests at 7 days for unit compressive strength					Tests at 14 days for unit compressive strength					Tests at 28 days for unit compressive strength				
	No overload	10 percent overload		20 percent overload		No overload	10 percent overload		20 percent overload		No overload	10 percent overload		20 percent overload	
		Strength	Strength ratio [2]	Strength	Strength ratio [2]		Strength	Strength ratio [2]	Strength	Strength ratio [2]		Strength	Strength ratio [2]	Strength	Strength ratio [2]
	p.s.i.	*p.s.i.*		*p.s.i.*		*p.s.i.*	*p.s.i.*		*p.s.i.*		*p.s.i.*	*p.s.i.*		*p.s.i.*	
Alabama	2,890	2,950	102	3,320	115	------	------	------	------	------	5,720	5,260	92	5,740	100
California	------	------	------	------	------	2,300	2,240	97	2,130	93	------	------	------	------	------
Delaware	4,010	4,020	100	3,760	94	4,280	4,310	101	4,120	96	4,730	4,640	98	4,550	96
District of Columbia	3,140	3,130	100	2,950	94	3,580	3,560	99	3,380	94	3,880	3,800	98	3,660	94
Florida	2,920	2,470	85	2,760	95	3,290	2,750	84	3,560	108	3,760	3,320	88	3,940	105
Kansas [1]	------	------	------	------	------	------	------	------	------	------	4,560	3,620	80	4,150	91
Michigan	------	------	------	------	------	------	------	------	------	------	4,630	4,920	106	4,610	100
Nebraska	4,160	4,000	96	4,320	104	------	------	------	------	------	5,560	5,390	97	5,300	95
Ohio 1	------	------	------	------	------	------	------	------	------	------	5,370	5,440	101	5,450	101
Ohio 2	------	------	------	------	------	------	------	------	------	------	4,300	4,050	94	4,790	111
Virginia 1	------	------	------	------	------	------	------	------	------	------	3,620	3,330	92	3,240	90
Virginia 2	3,740	3,690	99	------	------	4,260	3,990	94	------	------	4,440	4,170	94	------	------
Washington	------	------	------	------	------	------	------	------	------	------	------	4,630	------	4,660	------
West Virginia	2,940	2,560	87	------	------	3,640	3,410	94	------	------	4,240	4,100	97	------	------
Average	------	------	96	------	100	------	------	95	------	98	------	------	94	------	98

[1] Kansas tested cores at 28 days with the following results: No overload, 5,030 p.s.i.; 10 percent overload, 4,430 p.s.i.; 20 percent overload, 4,530 p.s.i.

[2] Ratio to strength for no overlo

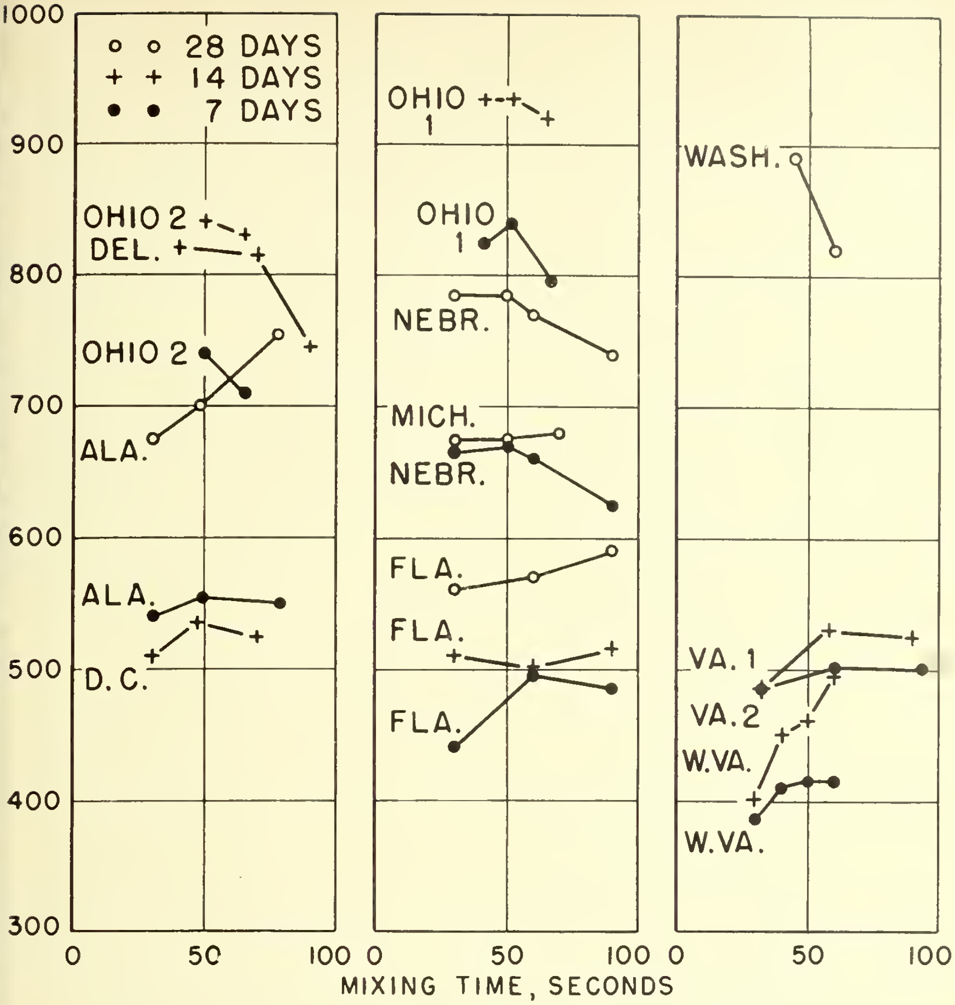


Figure 3.—Effect of mixing time on flexural strength, by State.

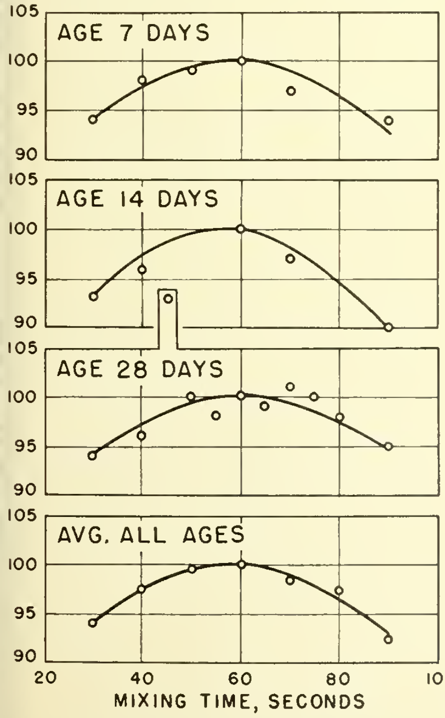
ight have influenced the general average an unwarranted extent and that they ight well be excluded from consideration.

To examine more carefully the results of the Florida tests, the compressive strengths obtained for each combination of mixing time

and overload were plotted against age at test. It was believed that if there had been any unusual features affecting the test results, these would be shown by some exceptional behavior in rate of gain of strength. As shown in figure 8, all of the Florida tests made for concrete prepared with 90 seconds mixing time produced an unusual behavior in rate of gain of strength. In two of the three cases, the strength at an age of 14 days was lower than that at 7 days. In the third case, the strength at 7 days appears to be quite low and that at 14 days somewhat high. Why these results had this unusual behavior is not known, but there is indication that they were caused by some feature other than the variables of record and it is believed that these results may justifiably be excluded from consideration. Consequently, in consideration of the effect of overload on compressive strength, all three sets of specimens mixed for 90 seconds in Florida were rejected.

Figure 9 presents revised trends of the effect of overload on compressive strength with the questionable and incomplete data excluded. Even with these data excluded, it was found that the strength for an overload of 10 percent was slightly lower than that for an overload of 20 percent, and both overloads furnished less strength than no overload. The maximum amount of reduction in strength is only 4 percent which, in view of the more rapid production of concrete gained by overloading, may be considered of minor consequence.

In the tests to determine the effect of overload on flexural strength, certain test specimens yielded irregular results, as shown in table 6 and figure 10. In no case, however, were all specimens for a given time of mixing involved, and all data obtained were used to determine average trends for all of the studies.



igure 4.—Average effect of mixing time on compressive strength.

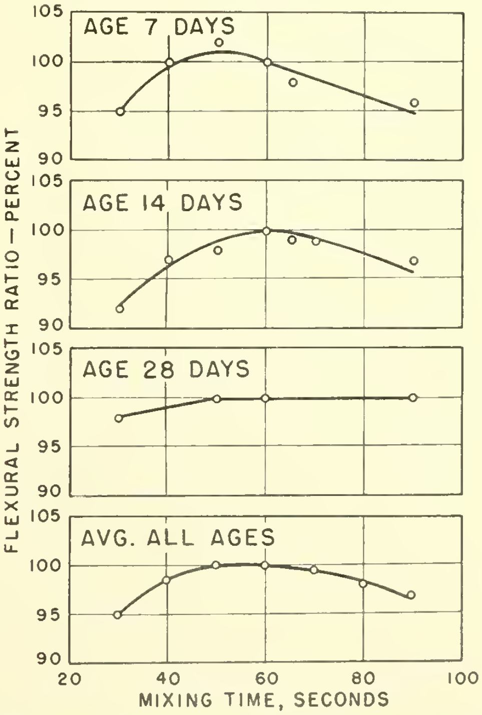


Figure 5.—Average effect of mixing time on flexural strength.

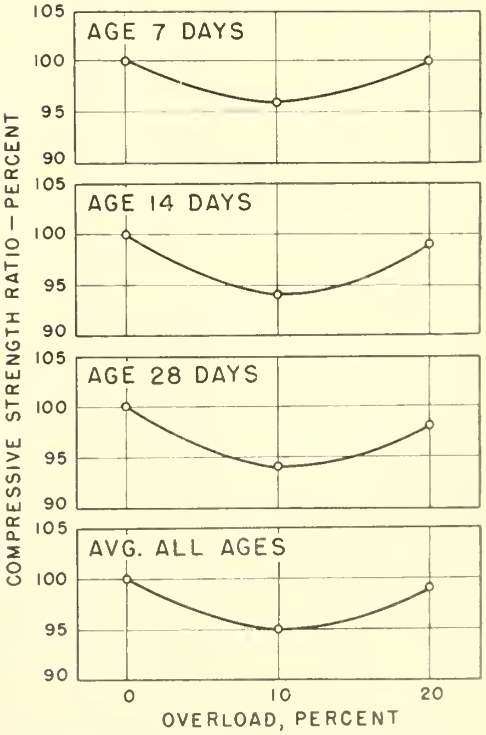


Figure 6.—Average effect of overload on compressive strength.

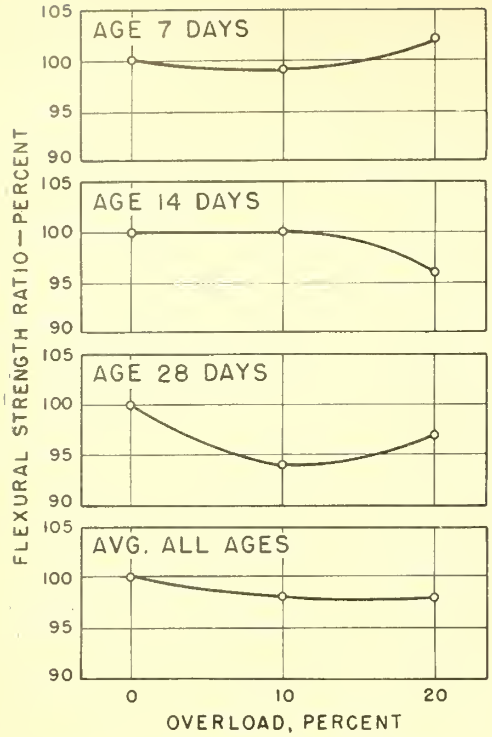


Figure 7.—Average effect of overload on flexural strength.

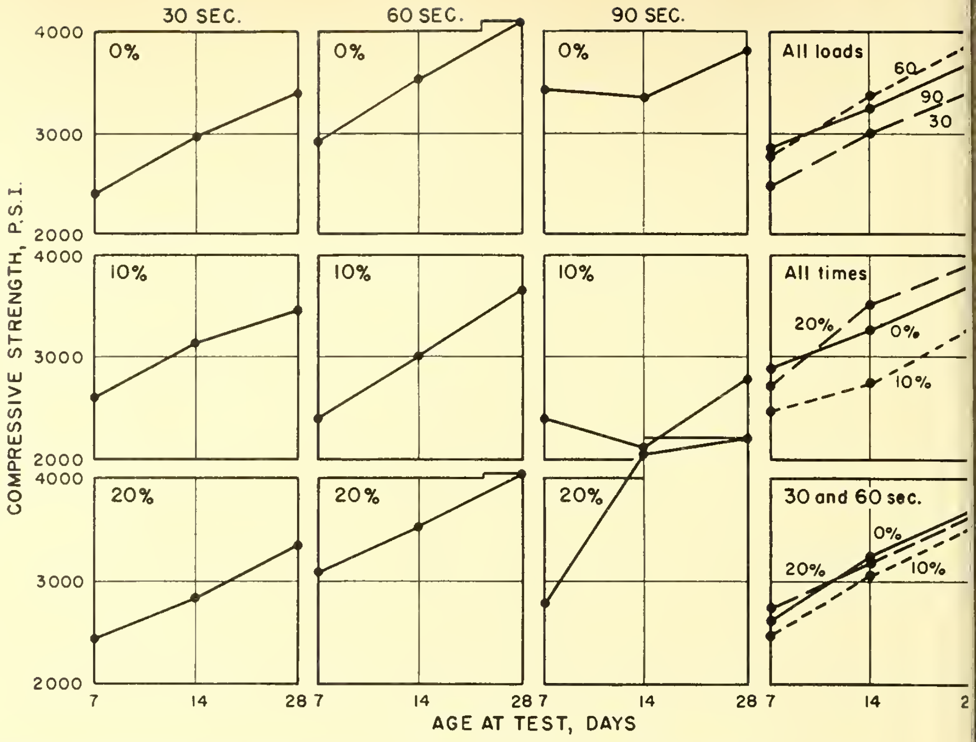


Figure 8.—Compressive strength for each set of variables using Florida test data.

Effect of Make of Paver

It was hoped that sufficient data could be obtained from the projects using the same make of paver to determine whether that make of paver processed concrete in a manner sufficiently different from the other pavers to warrant comment. It was considered desirable to limit the data used to that representing two or more projects and four mixing times.

The results obtained for a single make of paver showing the effect of mixing time on strength are shown in figure 11. By comparison with figures 4 and 5, it will be seen that there is little difference between the curves for this single make of mixer and the curves for all makes over the range of 50 to 70 seconds mixing time. It is probable that more difference in effectiveness mixing will be found between individu mixers of any make than between groups mixers of different makes. In two of the studies the mixers used were found, on initi inspection, to contain a considerable amou of hardened concrete. Removal of tl concrete unquestionably improved the effe tiveness of the mixers.

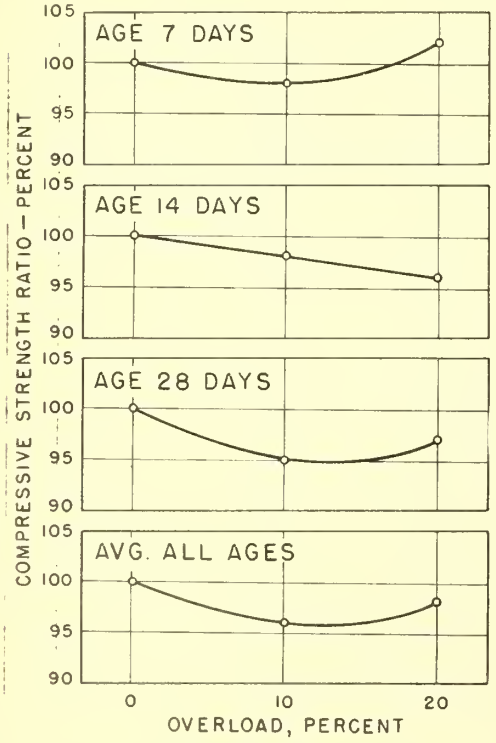


Figure 9.—Average effect of overload on compressive strength excluding questionable and incomplete data.

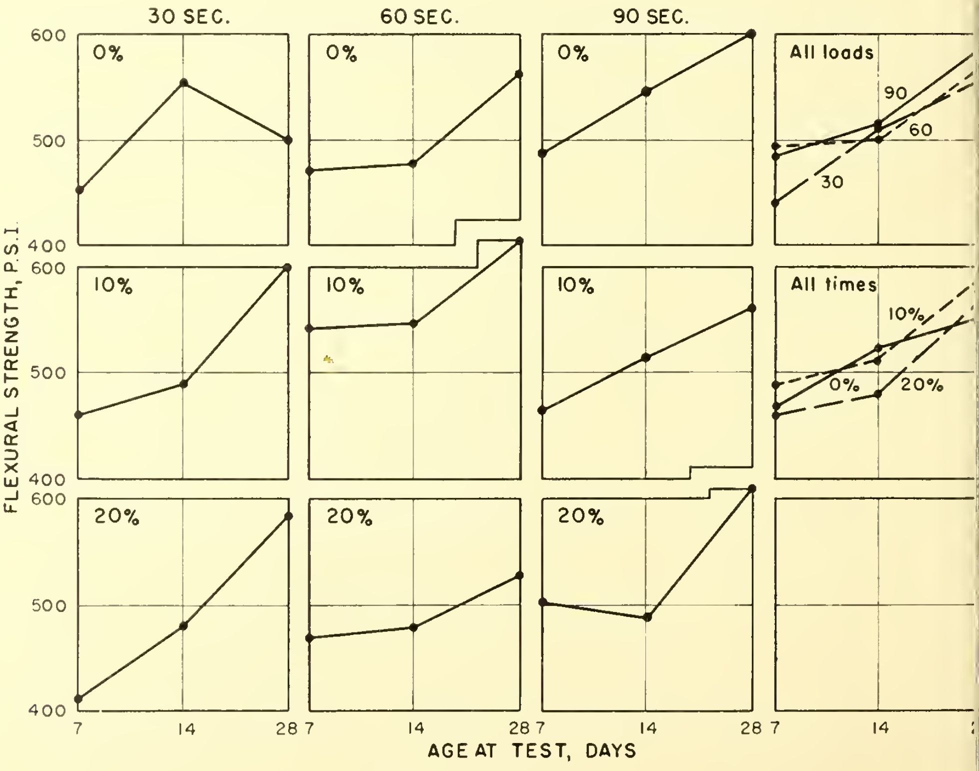


Figure 10.—Flexural strengths for each set of variables.

Table 6.—Effect of overload on flexural strength

State¹	Tests at 7 days for unit flexural strength					Tests at 14 days for unit flexural strength					Tests at 28 days for unit flexural strength				
	No overload	10 percent overload		20 percent overload		No overload	10 percent overload		20 percent overload		No overload	10 percent overload		20 percent overload	
		Strength	Strength ratio ²	Strength	Strength ratio ²		Strength	Strength ratio ²	Strength	Strength ratio ²		Strength	Strength ratio ²	Strength	Strength ratio ²
	p.s.i.	p.s.i.		p.s.i.		p.s.i.	p.s.i.		p.s.i.		p.s.i.	p.s.i.		p.s.i.	
Alabama	515	550	107	575	112	------	------	------	------	------	785	640	82	710	90
Delaware	------	------	------	------	------	795	820	103	760	96	------	------	------	------	------
District of Columbia	------	------	------	------	------	545	530	97	500	92	------	------	------	------	------
Florida	470	490	104	460	98	525	515	98	480	91	555	595	107	575	104
Kansas	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------
Michigan	------	------	------	------	------	------	------	------	------	------	680	665	98	680	100
Nebraska	680	640	94	640	94	------	------	------	------	------	810	735	90	765	94
New York	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------
Ohio 1	825	800	97	835	101	955	875	92	960	101	------	------	------	------	------
Ohio 2	720	730	101	730	101	840	845	101	820	98	------	------	------	------	------
Virginia 1	------	------	------	------	------	515	520	101	505	98	------		------	------	------
Virginia 2	505	485	96	------	------	------	------	- - --	------	------	------	------	------	------	------
Washington	------	------	------	------	------	------	------	------	------	------	------	865	------	845	------
West Virginia	415	400	96	------	------	430	470	109	------	------	------	------	------	------	------
Average ³	------	------	99	------	102	------	------	100	------	96	------	------	94	------	97

[1] California, Kansas, and New York reported no tests for flexural strength. [2] Ratio to strength for no overload. [3] The average ratio to strength for no overload at all ages was 98 for both 10- and 20-percent overload.

Tests for Uniformity of Plastic Concrete

It would be expected that insufficient mixing would be evidenced by marked differences in he results of tests of the three portions of a bucket of concrete, possibly by greater than normal weight per cubic foot, and by low air content. Excessive mixing would be expected to be detected by excessive fines in the concrete, and possibly by a low air content.

Almost all of the States made and reported test determinations which were intended to be used to show the uniformity of the plastic concrete. These tests included determinations of the cement content by the Willis-Hime and Dunagan methods, washout tests for fine or coarse aggregates, tests for unit weight of concrete, tests for consistency using the slump cone and Kelly ball methods, and pressure, volumetric, and Chace tests for air content.

The data furnished by the States were studied, and a selection of these data was made to determine the uniformity of the concrete produced with the various mixing times and overloads. For ease in presentation and comparison, these data are shown in figures 12–20. In each figure, sample data reflecting various characteristics of the concrete are shown with reference to individual combinations of mixing time and overload. Data for the first one-third of the bucket load of concrete are indicated by a circle; for the second third, the data are indicated by a cross; and for the last third, by a triangle. The solid lines in the figures connect the average value for a bucket load.

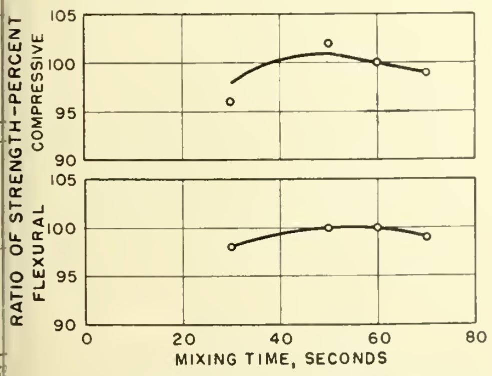


Figure 11.—Effect of mixing time on strength, determined from one make of paver.

Generally, the data plotted for each one-third portion is an average taken from three loads of concrete. These loads, obtained from different batches and on different days, could have been influenced by changing weather conditions, characteristics of the materials used, or by the personal equation of the operators. However, the sample loads indicate a general trend which may be considered adequate for this study and are not intended to cover all minute details which might influence such determinations.

For most of the reporting States, data covering tests for unit weight, washout, and air content were available. In the other States, data for slump were available.

Unit Weight

A marked variation in unit weight of concrete for a single batch of concrete or for several batches mixed under the same conditions would indicate either a harsh concrete or insufficient mixing. If a marked variation was found, and the same one-third portion generally had the greatest or least weight of the samples representing the same bucket load, it was assumed that the mixer was not operating properly and was furnishing undermixed concrete. For a given combination of aggregates, high unit weights, closely grouped, indicated well-mixed concrete.

General trends for all data from short to long mixing times were also of interest. If the trend appeared toward greater weight, an improvement in mixing was assumed. If the trend was toward lesser weight, this was taken to indicate overmixing and reduction in size of the aggregate.

Washout

The results of the washout tests were shown by several different methods. In some cases, the States determined and reported the amount of fine aggregate in the concrete or the grading and fineness modulus of the aggregate. In others, the amount of coarse aggregate was determined. Some States reported the grading of all aggregate, and choice had to be made as to which data would give the most information.

The results of these tests were expected to determine if extended mixing of concrete would abrade the aggregate to a marked extent. This could be demonstrated if there was a decrease in the fineness modulus of the fine aggregate, or an increase in the amount of fine aggregate and decrease in the amount of coarse aggregate. Undermixing of the concrete was indicated by the nonuniformity of the distribution of aggregate in the three portions of a bucket load.

Air content

The air content of the concrete was determined by use of pressure or volumetric methods, including the Chace air meter test as one of the latter methods. When available, data obtained with a pressure meter, or a volumetric method using a large sample of concrete, were preferred over data obtained with the Chace air meter.

The air content of air-entrained concrete was believed to be an excellent indication of the thoroughness of mixing the concrete. Insufficient mixing would be indicated by low amounts of air and by variation of the amounts of air between portions of a batch. With excessive mixing, it was believed that some of the air would be lost, but in this case the air in each portion of a batch would be practically constant.

Slump

The slump is a measure of the workability of concrete. A concrete can have a high water-cement ratio and at the same time have a low slump due to harshness of the mix. In very few instances were the concretes produced in these studies found to have poor workability, and it is believed that these cases can be ignored. The slump can be used with other data to deduce whether thoroughness of mixing was obtained.

Other determinations

Other determinations made on portions of a bucket load of concrete included the Willis-Hime test [1] for the cement content of concrete, and the Dunagan method [2] for determining the composition of concrete. The Willis-Hime test, made by the District of Columbia, New York and Virginia, yielded disappointing results in that they differed greatly from the known cement content. It was found that the principal reason for the results obtained was failure to dry the samples sufficiently. Since all of the water was not driven off, the cement grains still retained water and were not separated from the sand portion of the mortar during the centrifuging operation. This resulted in a test determination showing a low cement content of the concrete.

The District of Columbia made determinations of the composition of the fresh concrete using the Dunagan method. The results varied tremendously and indicated variations within and between batches of concrete which were beyond the realm of the possibility. Without doubt, some feature was overlooked in the performance of these tests which resulted in inappropriate test data.

Review of Uniformity by States

A review of the data for some of the individual State projects was made to determine information of interest with respect to uniformity of concrete. Such information is presented in the following paragraphs.

Alabama

The results from the Alabama project (fig. 12) showed a slight overall decrease in unit weight with increase in mixing time. With an overload of 20 percent, however, higher weights were obtained for each mixing time than for no overload. In 8 of the 9 combinations of mixing time and overload, the last portion of the concrete discharged by the bucket had the greatest weight. These facts collectively were assumed to indicate that the mixer was not operating to best advantage, and that an overload of 20 percent was an aid in obtaining better mixing of the concrete.

Determinations of the material passing the No. 4 sieve in the washout test showed some wide variations within a bucket load of concrete—especially for the 30-second mixing time. Each of the loads had an equal amount of sand, or more in the first one-third of the bucket load than in the second or last third portions. These variations indicated an insufficient mixing for a 30-second period. The general trend for all washout test results showed no increase in amount of fine aggregate with increase in mixing time and overload. This would indicate that even for a 90-second mixing time, there was no more abrasion of aggregates than for shorter periods of mixing time.

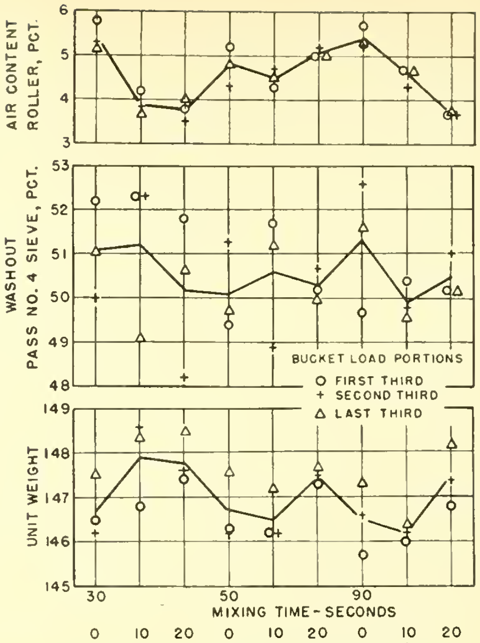


Figure 12.—Test determinations on uniformity of concrete in the Alabama project.

The air content tests, using a volumetric method, produced uniform values for the different portions of a bucket load. In two of the three mixing times, increase in loading resulted in a reduction in the amount of air. The air contents for a 30-second mixing time with no overload and with 20 percent overload were the same as those for the same loads at a 90-second mixing time. It would appear from these results that a 30-second mixing time is sufficient to obtain well-mixed concrete, and that further mixing is of little value.

Delaware

Data from the study in Delaware (fig. 13) showed no overall trend for unit weight except for a higher weight for each mixing time with a 10-percent overload. In 7 of the 9 cases, th weight of the last portion from the bucket wa equal to or greater than that of other portion This indicated that a greater percentage c stone was present in the last portion an further suggested that the mixer was no functioning properly.

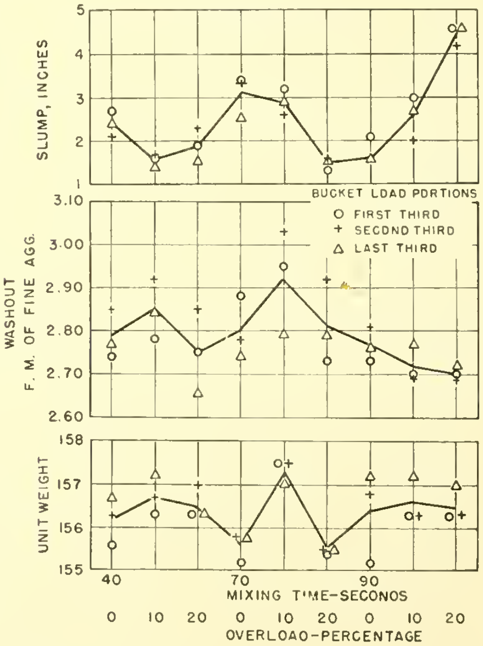


Figure 13.—Test determinations on uniformity of concrete in the Delaware project.

In the washout tests, the high value of fin ness modulus of sand for the 70-second mixin time with a 10-percent overload was matche by a similar high value for unit weight. Thes values were considered as sports, representin a nonuniform condition such as the use c unusually coarse sand. The other values fo the washout tests showed no unusual feature other than the prevalence of coarser sand fro the center of the bucket load and the pr gressive decrease in fineness modulus for th 90-second mixing time.

The progressive increase in fineness of th fine aggregate was almost matched by an i crease in the slump of the concrete for th batches involved. Although data for the wate content of the concrete were not immediatel available, the variations in the values fo slump were related to the variations in the uni weight values or the fineness modulus of th fine aggregate. Where an increase or decreas in slump occurred, a decrease or increase i unit weight was found or an increase or d crease in the fineness modulus of the san occurred. Considering all values given, th previously mentioned failure of the mixer t furnish well mixed concrete must be repeate It appeared that extended mixing of the co crete did abrade the aggregate to some exten It was also found that the first sample take from each bucket load generally had a lowe than-average unit weight, a finer-than-ave age sand, and a higher-than-average slump.

District of Columbia

The results obtained in the District o Columbia study (fig. 14) were unique in show

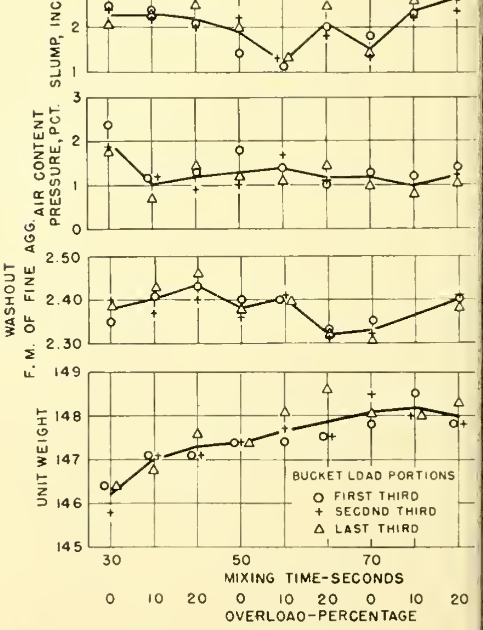


Figure 14.—Test determinations on uni formity of concrete in the District o Columbia project.

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[1] *Cement Content of Freshly Mixed Concrete.* ASTM Bulletin, No. 239, July 1959, pp. 48-49.

[2] *A Method of Determining the Constituents of Fresh Concrete,* by W. M. Dunagan. Proceedings, American Concrete Institute, vol. 26, 1930, pp. 202-210.

ng an increase in unit weight for each increase in mixing time and overload except for the 20-percent overload at 70-second mixing time. No corresponding trends were developed in the data for fineness modulus of fine aggregate, air content, or slump. It appeared that the uniformity of concrete was improved by increase in mixing time and load, the additional overload serving to promote the mixing action.

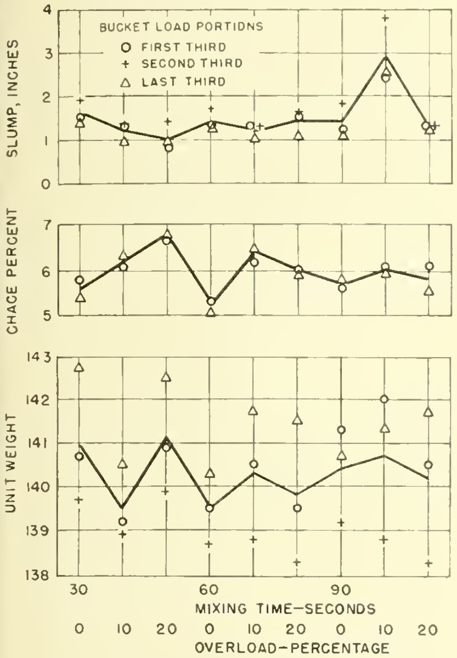


Figure 15.—Test determinations on uniformity of concrete in the Florida project.

Florida

The results obtained in the study in Florida (fig. 15) in some respects were quite unusual. Except for one nonconforming group of data, uniform results were obtained for the slump of concrete. The tests for air content made by the Chace method, also produced one sport, but otherwise the variations were insignificant. Neither set of data can be used to explain variations found in the tests for unit weight. In the concretes mixed for 60 and 90 seconds, those with a 10-percent overload had slightly greater average weights than those with no overload or 20-percent overload; while in the concretes mixed for 30 seconds, those with a 10-percent overload had the least weight.

In view of the marked variation in unit weight for the separate portions of all batches of concrete, and the fact that the last portion of each bucket load had the greatest weight in 7 of the 9 combinations of mixing time and overload, it appeared that the concrete mix was harsh and failed to respond adequately to increases in mixing time. A possible exception to this may be shown by the low slump. With a slump averaging only 1¼ inches, more variation in unit weight must be tolerated within a batch than for higher slump concrete.

Michigan

Data results obtained in the Michigan study (fig. 16), in which slag was used as the coarse aggregate, reflected in some respects the opinion that slag concrete is usually somewhat harsh. However, similar behavior was detected in data from other projects in which gravel was used.

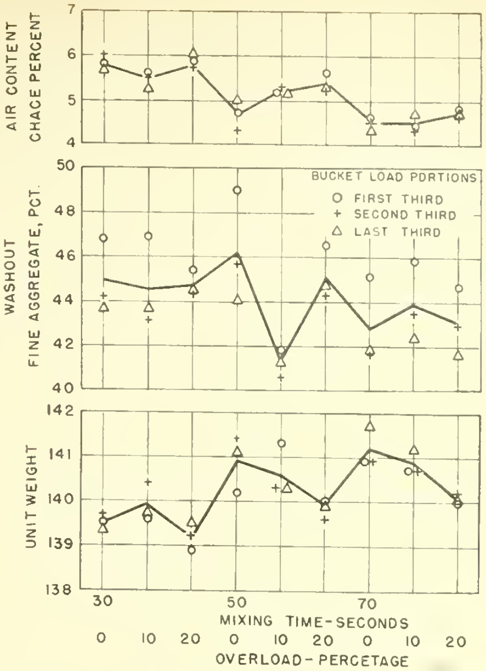


Figure 16.—Test determinations on uniformity of concrete in the Michigan project.

In the tests for unit weight, an increase in time of mixing was accompanied by an increase in the weight of the concrete. In the concretes mixed for 50 or 70 seconds, increase in overload resulted in a decrease in the unit weight, and in the concretes mixed for 30 seconds, those with a 20-percent overload had lower weights than those with no overload. These data indicate that an increase in mixing time and elimination of overload may furnish more uniformly mixed concrete for the particular mixer and concrete used in this project.

The washout tests showed a slight decrease in the amount of fine aggregate with an increase in mixing time and overload. The results of the air content tests showed a reduction in the amount of air with increase in mixing time and overload. This could mean that a 30-second mixing period was sufficient to develop maximum air content in the concrete, and that longer mixing periods permitted some of the air to be lost. It is noted that the data for air content and unit weight are associated. An increase of air content was associated with a decrease in weight, and selection of a most desirable set of mixing conditions definitely does not rest in these determinations.

Nebraska

For the study in Nebraska (fig. 17), the aggregate used was a mixture of sand-gravel and limestone. In conducting the washout tests, one determination made was the amount of crushed limestone found in the mixed concrete.

The tests for unit weight of concrete indicated some large variations both within and among batches. The general trend of these data indicated a slightly higher unit weight for concrete mixed for 60 seconds than for the other concretes included in the study. In 8 of the 9 combinations, the heavier concrete was found in the first portion of a bucket load, indicating that more large aggregate was contained in this portion than in the second or third portions. No relation between unit weight and air content results were found except for one batch mixed for 60 seconds with no overload. This concrete had the highest unit weight as well as the lowest air content.

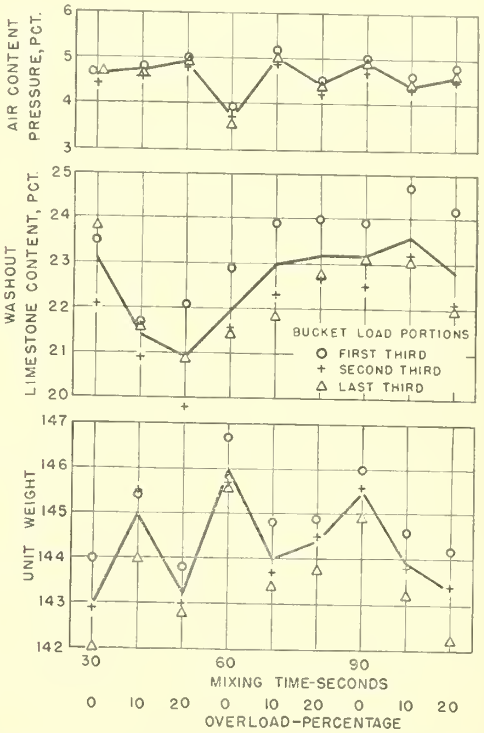


Figure 17.—Test determinations on uniformity of concrete in the Nebraska project.

The Nebraska report stated that a visual inspection of all test batches used did not disclose any poorly mixed concrete. This is of considerable importance as it may indicate that the variations shown in figure 17 may be of no significance with respect to concrete placed on the roadway.

Ohio project No. 1

In the first of the two studies conducted in Ohio (fig. 18), marked variations in unit weight were found, but the data showed no trends which could be associated directly with amount of mixing time or overload. The amount of coarse aggregate recovered from the concrete in the washout tests appeared irregular for the 40-second mixing time, and one group of results for the 50-second mixing time indicated a temporary lack of control at the batching plant. However, the results found for air content appeared to be quite uniform.

It was reported that inspection of the concrete as it was placed on the subgrade revealed only a few batches on which question might be raised regarding uniformity of mixing. Possibly uniformity in grading of the coarse aggregate and in batching the materials were of equal importance to the performance of the mixer.

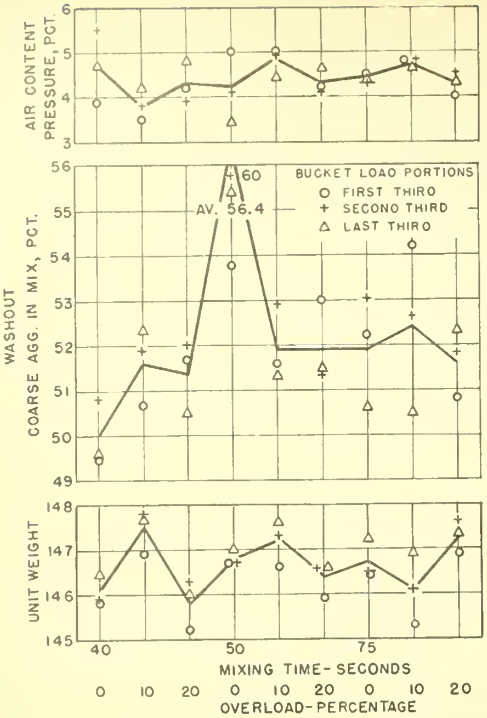


Figure 18.—Test determinations on uniformity of concrete in Ohio project No. 1.

Virginia project No. 1

It was reported for Virginia project No. 1 that the concrete was harsh and difficult to finish but this was corrected during the study. However, some of the variations of the data shown (fig. 19) may have been caused by this condition. Consequently, only a few comments on these data may be warranted.

It is interesting to note that the unit weight of the 30-second concrete was reasonably uniform, and that greater variations within batches were found for concrete mixed for 90 seconds. On the other hand, the air content for the 30-second concrete varied more from batch to batch than for the 90-second concrete. In the slump tests, the concrete mixed for 30 seconds with a 20-percent overload had a wide range between portions of the bucket load, but this could have resulted from use of the harsh concrete.

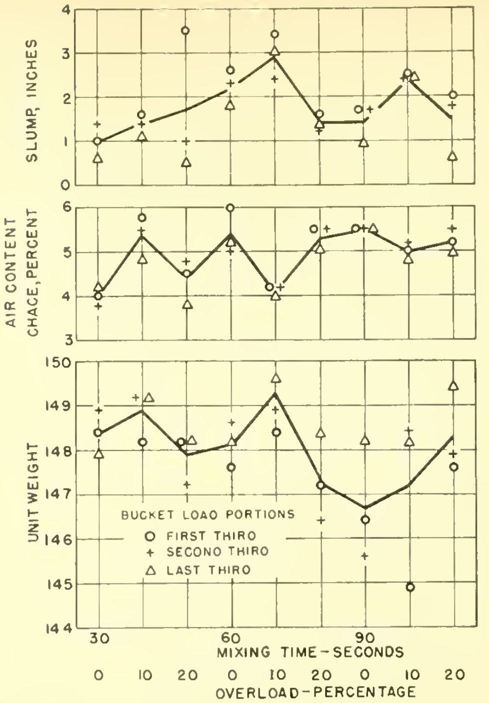


Figure 19.—Test determinations on uniformity of concrete in Virginia project No. 1.

West Virginia

The study in West Virginia (fig. 20) was hampered by cold weather and therefore some of the washout tests were not made. The notable variations in unit weight were in close agreement with corresponding variations in air content and slump. In general, the concrete mixed for 50 or 60 seconds was more uniform among batches than the concrete mixed for shorter periods, whereas very uniform concrete throughout a bucket load was found for all mixing times.

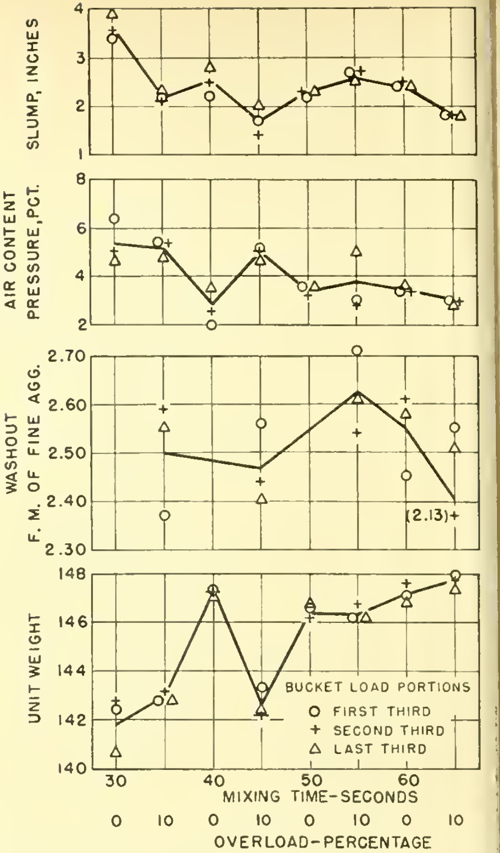


Figure 20.—Test determinations on uniformity of concrete in the West Virginia project.

Dual-Drum Paver Productivity: A Motion Picture

The Bureau of Public Roads, U.S. Department of Commerce, recently produced a new motion picture, *Dual-Drum Paver Productivity*. The film, based on field research studies conducted by Public Roads, provides evidence that many contractors can substantially increase job output for any given set of job conditions. The key lies in better control of job operating delays.

Action scenes from a variety of concrete paving jobs illustrate job conditions and performance factors encountered where paver productivity ranged from low on some jobs to outstanding on others. Illustrations show how batch plant efficiency and capacity as well as haul road variables affect the size of hauling fleet required to supply the paver. Numerous operating delays are highlighted to show how, and to what extent, they effect paver productivity. Actual job scenes show how these delays have been held to a minimum on high production jobs. The simultaneous showing of contrasting action scenes effectively highlights fast versus mediocre batch truck dumping performance at the skip.

Dual-Drum Paver Productivity is a 16-mm. sound and color film with a running time of 30 minutes. Prints may be borrowed for showings by any responsible organization by request addressed to Photographic Services, Bureau of Public Roads, Washington 25, D.C. There is no charge except for the express or postage fees. Requests should be sent well in advance of the desired showing, and alternate dates should be given if possible. Immediate return after each showing is necessary, so that all requested bookings may be fulfilled.

Prints of the film may be purchased at $122.83 per copy, the price including film, reel, can, and shipping container, and postage within the United States. Inquiries should be addressed to Photographic Services, Bureau of Public Roads, Washington 25, D.C. Payment should not be sent with the inquiry.

How a Dual-Drum Paver Operates

The accompanying series of sketches illustrates the functioning of a dual-drum paver—the machine generally used on-site to mix portland cement concrete for highway pavement—by following a single batch of cement and aggregate completely through the paver. The steps described below are keyed to the Roman numerals at the left of the sketches.

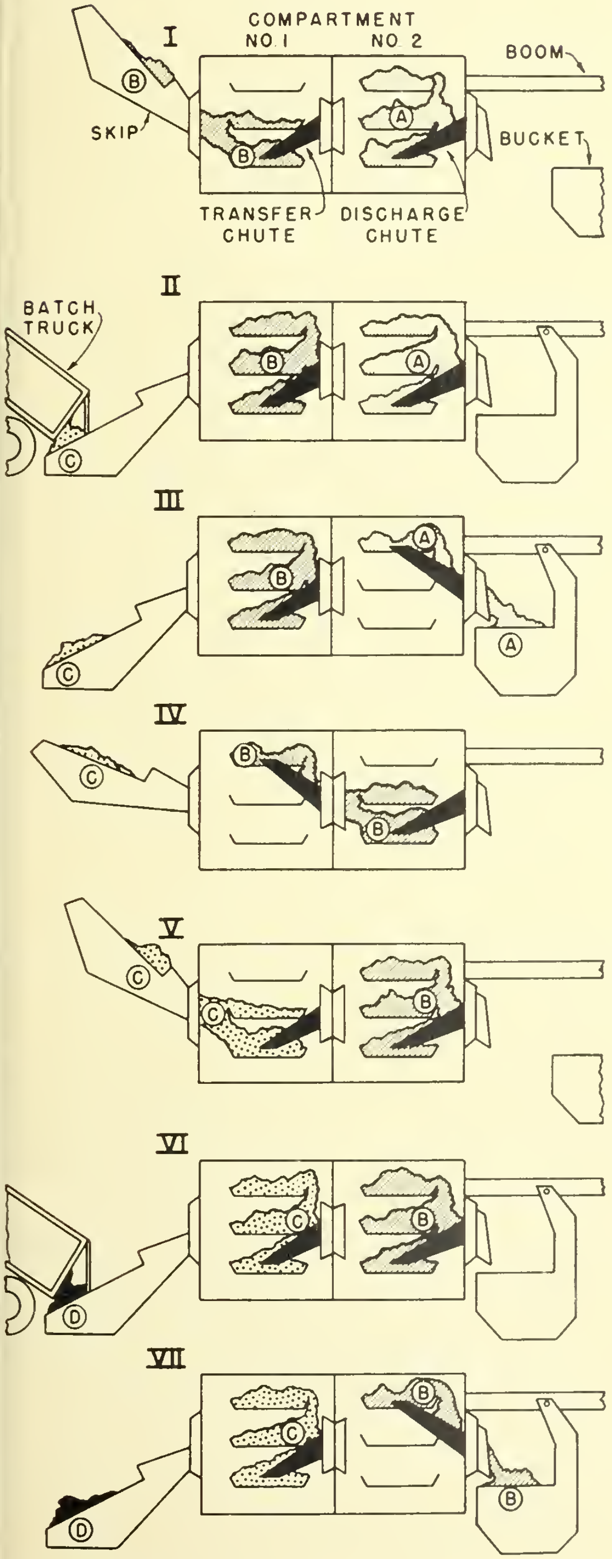
Step I.—The skip is up and batch B is entering compartment 1. Batch A, already in compartment 2, continues mixing. The transfer and discharge chutes are both closed. The empty bucket is returning along the boom.

Step II.—Batches B and A continue mixing in compartments 1 and 2, respectively. The skip is down and is receiving batch C from the batch truck. The empty bucket has returned. The transfer and discharge chutes are still closed.

Step III.—The discharge chute has opened, and batch A is being discharged from compartment 2 to the bucket. Batch B continues mixing in compartment 1, and batch C is in the skip, which is still on the ground. The transfer chute is still closed.

Step IV.—The discharge chute has closed and batch A is in the bucket, riding out the boom to be dumped. As the discharge chute closed, the transfer chute opened, and batch B is being transferred from compartment 1 to compartment 2. The skip is moving upward with batch C. These four steps complete one paver cycle.

Steps V, VI, VII.—These steps correspond exactly to steps I, II, and III, respectively. They are included in the series to show the progress of batch B through the paver from the skip, in step I, to the bucket, in step VII.



The batchmeter

The sequence of steps in the paver cycle is controlled by a batchmeter, a timing device which, at preset time intervals, actuates the successive operations of the paver or signals to the operator that certain operations can proceed. While there is a variety of types, most batchmeters have generally common principles.

With one popular type, the operator engages a control lever on the batchmeter (or keeps it engaged) to begin the paver cycle. The batchmeter then automatically starts the skip hoist, charges water into the drum, and closes the transfer chute. Next the meter signals when the discharge should be made, and the operator opens the discharge chute (if the bucket is ready). Then the meter, at the proper time interval, automatically closes the discharge chute and opens the transfer chute. As the discharge chute closes, and if the batch truck has emptied the next batch of material into the skip and moved clear, the operator engages the batchmeter control lever (or keeps it engaged) and the next cycle begins

With another popular type of batchmeter, the operator raises the skip which, on its way up, trips a lever which starts charging water into the drum, actuates the meter, and closes the transfer chute. After a preset time interval the batchmeter automatically opens the discharge chute, unless the operator intervenes because the bucket is not in position. Once the discharge chute has been opened, the batchmeter automatically closes it and opens the transfer chute. At the same time the meter signals to the operator that he can start the next cycle.

Mixing time

The time settings of the batchmeter theoretically control the mixing time which, for any one batch, is the sum of the times it spends in each of the two compartments. Mixing time is usually defined as beginning when all solid materials have entered the drum. This entry time takes from 3 to 11 seconds from the time the transfer chute closes, depending on the nature and moisture content of the aggregates. Some mixing specifications exclude the time of transfer of the batch from the first compartment to the second, which should take about 9 seconds.

The batchmeter cannot, of course, overcome operating difficulties which delay either charging or discharging. If, at the proper moment, the skip is not ready to go up, or the bucket is not in position to receive a mixed batch, the batchmeter timing device is stopped. When the batchmeter resumes, it runs out the full preset amount of the time interval that was interrupted. During the delay, however, the drum continues to turn on "unmetered" time. As a consequence, overtime mixing occurs, usually for the batch in the second compartment. Thus the true mixing time of any batch is the full preset time of the batchmeter plus any additional time occurring because of delays.

It should be noted that the batchmeter is a mechanical device working under adverse conditions of dust, moisture, and heavy vibration, and cannot be expected to parallel the precision timing of a jeweled watch. In addition, it cannot be ignored that the batchmeter's preset timing can be varied or nullified through actions of the paver operator, either inadvertent or deliberate.

Dual-Drum Paving: Cost vs. Mixing Time

BY THE HIGHWAY NEEDS AND ECONOMY DIVISION
BUREAU OF PUBLIC ROADS

Reported [1] by MORGAN J. KILPATRICK,
Chief, Construction Economy Branch

Skilled paving contractors have consistently produced portland cement concrete in dual-drum pavers with controlled mixing time of 45 seconds or less, including transfer time. The increased tempo required for operating the paver with a short cycle and the consequent short mixing time has been achieved in part through use of larger and faster batch trucks, and in part through the contractors' demonstrated awareness of the importance of eliminating unnecessary delays which consume valuable production time.

Monetary savings ranging from 10 cents to $3 per cubic yard of concrete might be realized if mixing time specifications were reduced to 45 seconds from their present levels of up to 120 seconds. In the current highway program, millions of dollars might thereby be saved annually.

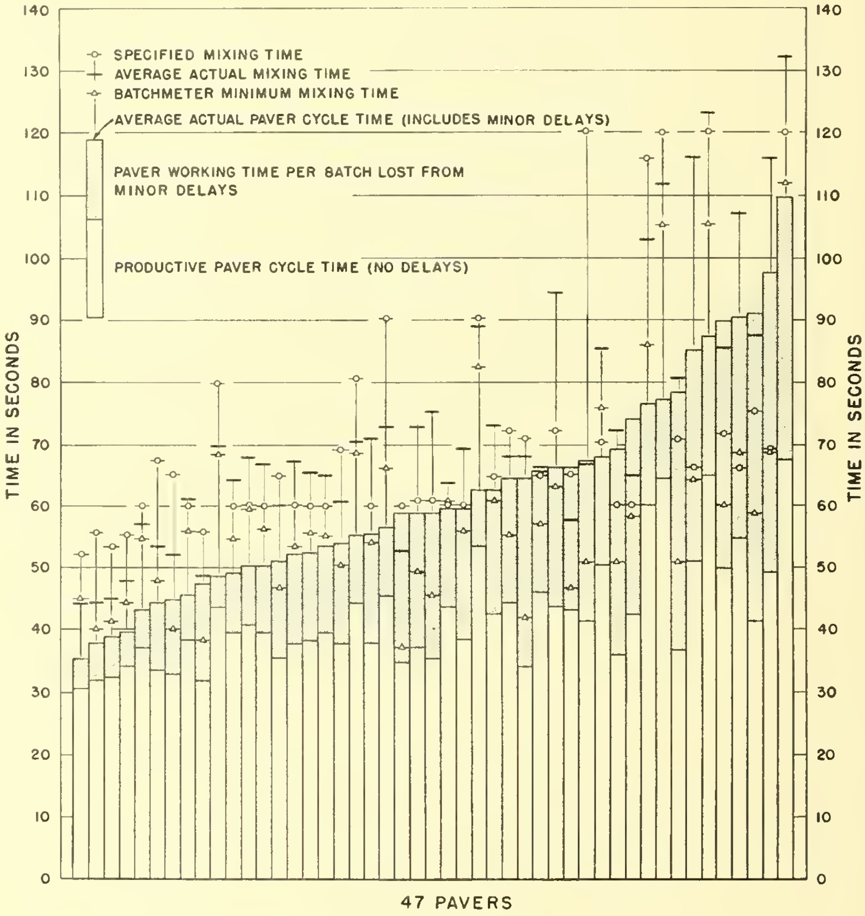


Figure 1.—Performance comparison of 47 dual-drum pavers.

Bureau of Public Roads on-the-job studies of the production of portland cement concrete for highway pavements indicate that some contractors, through adroit management and use of good and well-maintained equipment, achieve sustained high rates of production with dual-drum pavers. The fast paver cycle thus shown to be practical also results in a shorter mixing time for the concrete—less than is now specified by many contracting authorities.

To the engineer the fast cycle poses a problem—that of quality control, usually expressed in terms of mixing time. This is the subject of the article *Study of 34-E Dual-Drum Pavers,* appearing on page 1 of this issue of PUBLIC ROADS, in which it is shown that the longer mixing times now commonly specified can be appreciably reduced without affecting the quality of the concrete.

But there is another consideration involved in the fast cycle—that of economy, which is the subject of this article. Starting from a basis of factual knowledge, it will be demonstrated theoretically that reasonable reduction of specified mixing time will result in the production of concrete, by efficient contractors, with savings ranging up to as much as $3 per cubic yard. Related to the huge highway program now under way, such unit reductions in cost have the potential of saving millions of dollars annually.

Definitions

For clarification, the following definitions are offered. The reader is also referred to the short descriptive article, *How a Dual-Drum Paver Operates,* on page 11 of this issue.

Paver cycle.—The paver cycle is the sequence of paver operations from one charging of the drum to the next.

Skip cycle.—The skip cycle includes those steps of the paver cycle in which the skip is raised, the batch is charged into the drum, and the skip is lowered to the ground where it is then ready to receive the next batch of material dumped by the batch truck.

Mixing time.—As used in this article, mixing time is the time interval from complete entry of a batch of cement and aggregate into the drum to the beginning of discharge of the same batch into the bucket, including transfer time between the two compartments of the drum. (In the article on p. 1, for reasons explained therein, transfer time was excluded from mixing time.) For any given paver cycle, there is a maximum mixing time that can be obtained when the paver is adjusted and operated in a manner consistent with the manufacturer's recommendations. In this article it is assumed that paver operations are in accord with such recommendations.

Key to Fast Paver Performance

In the production of portland cement concrete with dual-drum pavers, a short paver cycle means short mixing time and reduction of the time available in which to perform essential operations at the paver. When the paver cycle is extended by delays a longer mixing time inevitably results, thus nullifying the timesaving advantages of operating with a short cycle. Most delays become apparent at the paver skip and it is here that one can find not only a place to measure qualitatively the contractor's management but also the clue to interdependence of mixing time and batch-truck performance.

The key to fast paver performance thus centers around the ability of the paver operator to run a short skip cycle, the batch-truck driver to dump his batches into the skip on schedule, and to a lesser degree on the coordination among these and other operations in front of and behind the paver. Delays resulting from careless or sluggish performance by either the paver operator or the batch-truck driver, and to a lesser extent by others, can effectively stifle the orderly flow of material, extend the paver cycle, and disrupt both supporting and dependent operations.

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[1] This article was presented at the 39th annual meeting of the Highway Research Board, Washington, D.C., January 1960.

If the paver operator is slow in lowering the skip, the time remaining for the batch-truck driver to dump his next batch in the skip may not be enough to prevent a delay from occurring, even though he may perform his job well. If, on the other hand, the batch-truck driver fails to keep his truck close to the skip between batches, does not back up promptly when the skip comes down, or fails to have the dump bed raised or at least started up before backing to the skip, he may be too slow in disposing of his batch. The paver operator will then be delayed in raising the skip to begin a new cycle. In each case the paver cycle is extended by the delay, and mixing time is thereby increased.

Many contractors have successfully mastered the problems of getting essential coordination and performance at the skip, as well as keeping attendant supply lines moving. On one markedly successful operation, sustained production by the paver was recorded at a rate of 98 batches per hour. For intermittent periods lasting up to 2 hours, a 110-batch rate was attained. In other words, a batch of concrete was discharged by the paver every 33 seconds, including delay time. When a small number of extended delays, due to lack of batch trucks, was eliminated from the timing, production was at the remarkable rate of one batch every 30 seconds. Remember, this was an average for extended periods and included minor delays. When no delays occured, individual paver cycles were being completed in 24 seconds or even less, with a mixing time of about 27 seconds.

The operation just cited may seem remarkable, but it indicates what can and is being done. Figure 1 shows the performance data for 47 dual-drum pavers on which production studies, usually of 2–4 weeks' duration, were made by the Bureau of Public Roads. Note that each of the four jobs with the fastest performance had a job average cycle time, including minor delays, of less than 40 seconds. It will be seen that on many other jobs batchmeter settings were sufficiently short that a cycle time of less than 40 seconds could have been achieved if the minor delays had been reduced or eliminated.

Relation of Mixing Time to Paver Cycle

The design of the dual-drum paver is such that two batches are mixed simultaneously during a portion of each paver cycle. The mixing time that can be obtained with any given paver cycle is expressed by the formula:

Mixing time $= 2X - D - T - C + L$

Where:

X = paver cycle time.

D = discharge chute open time (typically 9 seconds).

T = transfer chute open time (typically 9 seconds).

C = charging lag time, from close of transfer chute when skip approaches vertical until all solid material is in the drum (job averages of 4 seconds have been attained with dry aggregates).

L = discharge lag time, from opening of discharge chute until concrete appears (averages 1 second).

Obviously, any increase in the time taken for the three elements, D, T, and C, will reduce mixing time in relation to the paver cycle time.

As an example of use of the formula, with a 33-second paver cycle and the indicated values:

Mixing time $= (2 \times 33) - 9 - 9 - 4 + 1 = 45$ seconds.

As previously noted, performance records have shown that on at least one job paver cycles could be completed regularly in 30 seconds which, according to the formula, provided a maximum mixing time of 39 seconds. With the occasional 24-second cycle attained on the job, the mixing time was 27 seconds.

Requirements for a Fast Paver Cycle

The very fast paver-cycle values and consequent mixing times just cited represent actual performance of a single, rather unusual job. An analysis of the performance requirements for maintaining a short paver cycle, based on averages obtained on a number of fast-moving jobs, demonstrates what is fully practical of achievement.

During the paver cycle the skip must be raised, the mixing drum charged, the skip returned to the ground, and then the skip reloaded. This skip cycle took an average of 20 seconds on all jobs studied and often was accomplished regularly in 15 seconds, but seldom in less. The difference in length of time between the skip cycle and the full paver cycle is available for reloading the skip. To do this, the batch truck must back up and cover the skip, dump its batch, and pull away. The paver operator must see that the truck is clear before starting the skip up to begin a new paver cycle.

It was found, in the Public Roads studies, that batch trucks on fast-moving jobs consistently backed into position to dump into the skip in 2 or 3 seconds. As shown in figure 2, it was observed on a very fast job (curve A) that the trucks dumped and pulled away in 3 to 5 seconds. On several other fast jobs (curve B) up to 85 percent of the batches were dumped in less than 8 seconds. It should be noted, however, that such performances were attained only with 4- and 5-batch trucks. The best performance with 2-batch trucks (curve C) took 15 seconds or less for 85 percent of the batches. On other fast jobs the 2-batch trucks took 20 seconds or less for 85 percent of the batches (curve D).

It was also found that after a batch truck cleared the skip, the paver operator took about 2 seconds before he actually started the new cycle.

Combinations of these figures represent possible paver cycle time, from which mixing time can be derived. For example, really fast

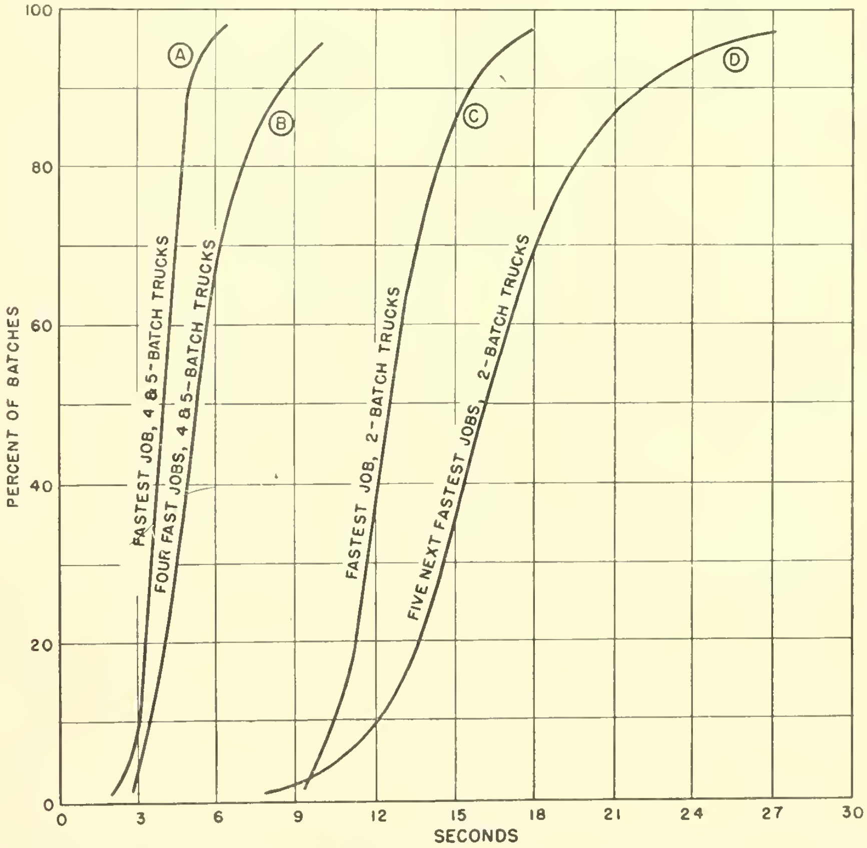

Figure 2.—Cumulative frequency distributions of average time to dump one batch into the paver skip and get the truck out.

operation would include 3 seconds for backup, 5 for dumping, 2 for operator's reaction, and 15 for the skip cycle, totaling 25 seconds for the paver cycle and producing a 29-second mixing time. A more representative fast job using 4-batch trucks, with 8 seconds for dumping and 17 for the skip cycle, would have a 30-second paver cycle and a 39-second mixing time.

With 2-batch trucks the fastest dumping was 15 seconds which, with the 15-second skip cycle time, gives a 35-second paver cycle and a 49-second mixing time. More representative of typically fast operation with 2-batch trucks is an 18-second dumping time and a 17-second skip cycle, producing a 40-second paver cycle and a 59-second mixing time.

Long Cycles, Long Delays

Two methods have been used in this article in arriving at a minimum paver cycle time and the resulting mixing time. In the first, data recorded on one very fast operation revealed that a paver ran for a 2-hour period with an average paver cycle of 30 seconds (excluding time lost in a few extended delays), permitting a mixing time of 39 seconds. Mixing time was further reduced to 27 seconds for individual batches.

In the second approach, paver cycle time and mixing time were derived from performance data obtained from a number of well-operated jobs. A 30-second cycle was found to be entirely practicable and a 25-second cycle was attainable, with 4-batch trucks. These cycles produced mixing times of 39 and 29 seconds, respectively. However, no allowance was made for delays in the calculations.

In considering this problem of delays, it was generally found that more time per batch was lost through delays by pavers with a long paver cycle than was lost by those with a short paver cycle.

This paradoxical fact is evident in figure 3. Curve A in the figure represents the average paver cycle time of each of the 47 jobs studied, including minor delays, while curve B represents paver cycle time excluding delays, averaged in groups of five jobs. Curve C, the difference between the values of A and B, represents the delay time per cycle. It will be noted that the longer the paver cycle time, the greater was the amount of delay, and that conversely the jobs with the fastest cycles had the smallest amount of delay.

The most logical explanation for this seemingly paradoxical trend is that as the permissible production potential increased with shorter paver cycles, management became more alert and responsive to the needs and possibilities for reducing or eliminating delays.

Cost Factors

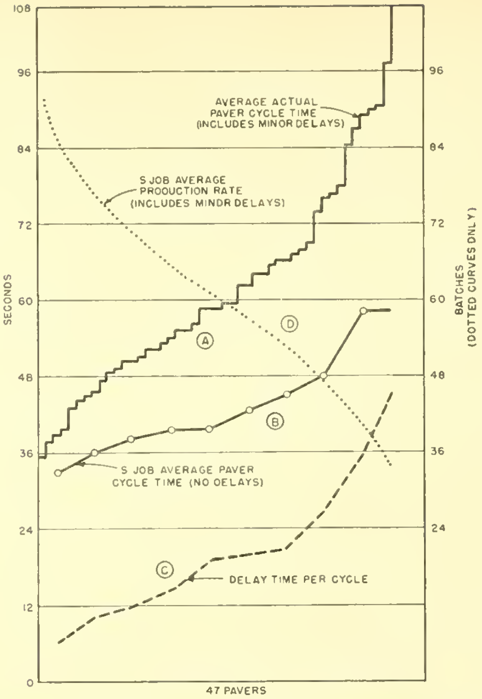

Figure 3.—Comparison of average paver cycle times, including and excluding delay time, for 47 dual-drum pavers.

It is an inescapable fact that delays increase costs. With equal assurance it can be said that mixing time requirements influence costs. The cost of operating a paver outfit tends to be fixed and is almost independent, within reasonable limits, of the rate of production. Labor and equipment costs amount to about $350 per hour, on the average. When production is 50 batches per hour, then, the cost is $7 per batch. However, if production goes up to 70 batches per hour, which is entirely practicable with good batch-truck performance, the unit cost drops to $5 per batch. Thus, with such production improvement, the cost reduction per batch is $2. This amount is a function of the percentage increase or decrease in production and the per-batch difference will diminish as additional batches are added to the hourly production rate. Thus, an increase of 40 batches per hour, from 50 to 90, results in a unit cost of $3.90 per batch, and a reduction in cost of $3.10 per batch. In other words, the first increase of 20 batches is worth $2 per batch, but in the increase of 40 batches the second 20 are only worth $1.10 per batch.

Data presented in figure 3 also show that hourly production rates, including time lost in minor delays (those of less than 15 minutes each), go down from a high of about 92 batches to a low of 35 per hour as average paver cycle time is increased.

Using data from which figure 3 was derived, for only those pavers which had 50- and 120-second mixing time specifications in force, the production rates averaged 85 and 40 batches per hour, respectively. However, these rates were reduced to 75 and 34, respectively, when the time basis includes certain major delays (those in excess of 15 minutes each) which occurred while a full labor force was employed on these jobs. Using a fixed cost of $350 per hour, the cost differential between 75 and 34 batches per hour becomes $5.63 per batch.

Effect of overload on cost

With a fairly common overload of 10 percent, resulting in a 37.4-cubic-foot batch, the cost differential becomes $4.06 per cubic yard of concrete. Further adjustment is desirable, under certain circumstances, to account for the variation in batch sizes used for different mixing times. In instances where 50-second mixing time is used a 20-percent overload is usually allowed. This will yield a 40.8-cubic-foot batch. Thus, the difference in production cost becomes $4.34 per cubic yard when comparing 40.8-cubic-foot batches at a rate of 75 batches per hour with 37.4-cubic-foot batches at a rate of 34 batches per hour.

Effect of increased production on cost

It may be reasonably assumed that hourly production costs are somewhat less when producing 34 batches per hour than when producing 75. The effect of this consideration is to reduce the cost differential. It could be assumed, for example, that the $350 per hour cost used is for a rate intermediate between 75 and 34 batches per hour, and that the cost would be 10 percent less for the 34-batch rate and 10 percent more for the 75-batch rate. The differential in production cost between the two rates, then, instead of being $4.06 per cubic yard, would drop to $2.98 for 37.4-cubic-foot batches.

It would be impractical to obtain specific cost data for every possible rate of output and, in fact, since a number of assumptions must be made, it would be pointless for the purpose here intended. The unit costs developed above, while based on some factual information and reasonable assumptions, are admittedly hypothetical. However, the method does establish a means for computing production cost differentials due to variations in mixing time specifications.

Data from Federal-aid records

Another source of information on this subject is the data compiled from bid prices per cubic yard and specified mixing times for portland cement concrete placed on Federal-aid projects.

The fairly uniform relation of the two is shown (for the year 1957, the latest available) in figure 4. It will be noted that from a low bid price of $15.75 per cubic yard for 50-second specified mixing time there is a gradual increase of both price and mixing time up to $23.15 for 120-second mixing—a difference, from low to high, of $7.40 per cubic yard. The difference is more apparent than real, however. If it actually is a true indication of the differential resulting from the range in mixing time, then it also means that the labor and equipment cost of running a paver outfit exceeds $700 per hour. Such an hourly cost is unlikely, if not impossible, and cannot be accepted at face value as representing an average.

A more plausible explanation of the $7.40 differential appears to be that part of it could be charged to the mixing time differences and that the balance represents costs generated by regional differences in design practices, hauling distances, and variations in labor productivity, wage rates, climatic conditions, and materials costs. The last mentioned, in particular, influence bid prices but not paver production costs. It is perhaps not without

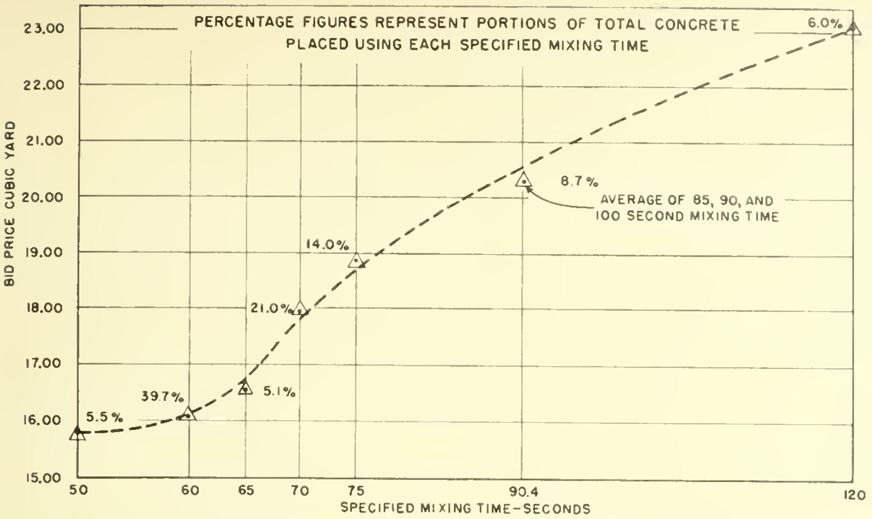

Figure 4.—Distribution of bid price per cubic yard of concrete production according to specified mixing times on Federal-aid projects in 1957.

significance that most States specifying longer mixing times are in the Northeast, whereas those using shorter mixing times are in the South and Southwest where year-round work is possible.

Calculation of Potential Savings

The basic objective of this article is to develop quantitatively the potential savings which might result from reduction in mixing time. It is believed that the fixed cost of $350 per hour for labor and equipment, discussed earlier, offers the most plausible starting point. To allow for variance in production in relation to mixing time, the previously discussed assumptions were used: Production of 34 batches per hour with a 120-second mixing time would cost 10 percent less than the $350 base; production of 75 batches per hour with a 50-second mixing time would cost 10 percent more. Intermediate values were taken from a straight-line projection through these points.

With these premises, the production costs per cubic yard of concrete were calculated for a series of mixing times. These are shown in the first two columns of table 1. It was then a simple matter to derive the approximate production cost reductions that are possible by decreases in mixing times, shown in the remainder of table 1.

It should be remembered that the mixing times shown in table 1 include transfer time. For comparisons with data in the article on page 1, which exclude transfer time, about 10 seconds should be subtracted from mixing times used in table 1. As pointed out in the article on page 1, mixing time of 40 seconds produced concrete with very little strength loss as compared with 60-second mixing; and 30-second mixing time resulted in a strength loss of 5 or 6 percent. In terms of table 1, these values would be 50- and 40-second mixing times, compared to a 70-second base.

Table 1.—Production cost[1] per cubic yard of concrete according to mixing time, and theoretically possible cost reduction resulting from decreases in mixing time

Base mixing time *(seconds)*	Production cost per cubic yard	Cost reduction per cubic yard possible by decreasing base mixing time to—								
		90 seconds	75 seconds	70 seconds	65 seconds	60 seconds	50 seconds	45 seconds	40 seconds	35 seconds
120	$6.69	$1.85	$2.36	$2.51	$2.64	$2.77	$2.98	$3.08	$3.17	$3.25
90	4.84	------	0.51	0.66	0.79	0.92	1.13	1.23	1.32	1.40
75	4.33	------	------	0.15	0.28	0.41	0.62	0.72	0.81	0.89
70	4.18	------	------	------	0.13	0.26	0.47	0.57	0.66	0.74
65	4.05	------	------	------	------	0.13	0.34	0.44	0.53	0.61
60	3.92	------	------	------	------	------	0.21	0.31	0.40	0.48
50	3.71	------	------	------	------	------	------	0.10	0.19	0.27
45	3.61	------	------	------	------	------	------	------	0.09	0.17
40	3.52	------	------	------	------	------	------	------	------	0.08

Admittedly, it is a bold step to apply the theoretical calculations in table 1 to an entire highway program, but by so doing the point can be driven home. The distribution of concrete production according to specified mixing time on Federal-aid projects (in 1957) is shown in figure 4. It is worth noting, incidentally, that the weighted average of all mixing times was 70 seconds. In the Federal-aid Interstate and primary highway programs, 18 million cubic yards of pavement concrete were laid in 1958, and 20 million would seem a conservative figure to use now.

Working from these bases, the theoretical annual savings in the Federal-aid Interstate and primary programs that would result from reductions in mixing time can be computed. In the first place, if only all higher than 70-second mixing times were reduced just to the 70-second level, the resultant annual savings would total about $4½ million; reduction of all higher mixing times to 60 seconds would save nearly $7½ million. If all mixing times higher than 50 seconds were brought down to the 50-second level, which would result in very little concrete strength loss, the total annual savings would be over $11 million. Similarly, for reduction of all higher mixing times to a 40-second mixing time, which would result in a strength loss of 5 or 6 percent, the savings would total $15 million. It will be noted that the reductions of higher mixing times by 10-second intervals produced increasing increments of savings. Further reductions in mixing time, however, would bring diminishing returns in savings. Such further reductions might also bring unacceptable reductions in concrete strength.

There is no question that the assumptions used here are speculative, although they are based on foundations of fact and considered judgment. There is also no question that a considerable proportion of the contracting industry is capable of converting into increased production the time that would be saved by shorter specified mixing times than are now prevalent. The rate of production is to a large extent governed by the performance of the paver operator and the batch-truck driver. Four-batch trucks, able to maneuver quickly and safely with fully loaded beds in hoisted position, are essential for shorter paver cycles, and they are coming into common use. It can be said that the industry is ready for shorter paver cycles, and resulting shorter mixing times of 50 seconds or less. It is encouraging to note that some States have recently reduced their specified mixing times, and others are now contemplating such changes.

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[1] Calculated production costs for labor and equipment. Materials not included.

Time and Fuel Consumption for Highway-User Benefit Studies

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported[1] by PAUL J. CLAFFEY,
Highway Research Engineer

Travel time and fuel savings are two of the benefits that accrue to highway users through highway improvement. Timesavings result whenever highway improvements reduce travel distance, permit higher speeds, or reduce the frequency of speed changes. Fuel savings are effected when improvements reduce travel distance, mitigate any of the resistances encountered by moving vehicles, or reduce the frequency of stop-and-go and slowdown operations.

During 1959 a study of passenger cars and single-unit trucks was conducted to determine the effect of variation of highway surface type and operating speeds on fuel consumption, and the effect of the elimination of both a stop-and-go and a slowdown operation on fuel and time consumption at various operating speeds. The study also included determination of the fuel consumed while vehicles were stopped with engine idling. Graphic and tabular material developed from the study and presented in this article will serve in estimating time and fuel benefits accruing through highway improvements.

BECAUSE there was a lack of complete data on the variation in time and fuel consumption for all vehicle types and weights, needed for use in highway-user benefit studies, the Bureau of Public Roads undertook a study in 1959 to determine the effects of highway surface conditions and traffic operations on passenger cars and single-unit trucks. The results of that study, arranged for practical use, are presented in this article. Some of the more general findings, which will also be of interest, are reported in the following paragraphs.

General Findings

Much of the savings in time and fuel enjoyed by users as a result of highway improvement arises because of increased vehicle speeds, smoother riding, and reduction of the frequency of stop-and-go and slowdown operations. On paved surfaces the rate of fuel consumption of passenger cars and single-unit trucks (fig. 1) was found to vary inversely with speed for the slower speeds, with the optimum at 25–35 miles per hour, varying according to vehicle type and gross weight. At higher speeds the rate of fuel consumption increased. On gravel surfaces the relation between rate of fuel consumption and speed (fig. 2) was similar, except that the lowest rate of fuel consumption was at 20–25 miles per hour.

The effect of upgrading from gravel to concrete on the rate of fuel consumption (fig. 3) increased with vehicle speed. At speeds of 15 miles per hour the increase in fuel consumption for the gravel surface was less than 7 percent, but at 45 miles per hour it was over 20 percent for passenger cars and pickup trucks and over 30 percent for single-unit trucks with gross weights of 10,000 pounds or more.

Both the additional time and fuel consumption for stop-and-go operations (figs. 4 and 5) increased uniformly with speed. At any speed the excess time consumption varied directly with weight-horsepower ratio. The additional fuel consumption varied with gross weight for single-unit trucks. The passenger car used in the study consumed more fuel than the heavier pickup truck at all speeds, probably because it was equipped with an automatic transmission while the pickup truck had a manual transmission.

The time consumption for a slowdown of 10 miles per hour (figs. 6 and 7) varied inversely with vehicle speed. The additional fuel consumption of passenger cars for slowdowns varied directly with speed for speeds up to 50 miles per hour, while for single-unit trucks it increased with speed up to speeds of 35–50 miles per hour but decreased somewhat at higher speeds.

Time and Fuel Savings

A major objective of highway-user benefit studies is the evaluation of advantages accruing to users as a result of highway improvements. Two of the more important of these advantages are reduced fuel consumption and reduced travel time. The relation between motor vehicles and the roads they travel is so close that even small changes in the characteristics of the road are reflected in the amount of time and fuel needed for highway trips. Minimum values of travel time and motor-fuel consumption are possible only when the roadway is ideally suited for the vehicle and to the traffic volumes in which the vehicle operates.

The ideal highway for such purposes would be straight and level, have a smooth surface, and be so designed that the movements of each vehicle would be completely unaffected by the presence of other vehicles. Although in practice no highway would be built to such improbable standards, all improvements are directed toward making highways less remote from this ideal. The improvement of a highway enables highway-users to complete their trips in less time and frequently with less fuel consumption. Highway-user benefit analyses, if they are to be complete and accurate, must include consideration of these savings.

Travel time savings are brought about through changes in highway facilities which reduce three elements: travel distance, the number of speed changes, and the time lost at traffic signs and signals. Time savings are also brought about by improvements which permit vehicles to be operated safely at higher speeds. Every mile of travel distance eliminated from the highway-user's trips saves time. Elimination of speed changes, such as stops and slowdowns, saves the time consumed while decelerating and accelerating.

Nominal highway speed

Nominal highway speed is defined as the modal operating speed of all vehicles of a given class while moving on sections of a highway where they are not slowed or stopped by highway impedances such as traffic signs and signals, sharp curves, etc. Highway changes which improve sight distance or add to highway capacity generally result in increased nominal highway speeds. On any road carrying a traffic volume equal to or greater than its practical capacity, the nominal highway speeds of vehicles will be increased by providing greater capacity through lane widening or additional lanes.

The nominal highway speed for vehicles having high weight-horsepower ratios will be increased mainly through reduction of grades.

Reduction of resistance factors

All improvements which lessen travel distance and the resistances to movement at onstant speed, plus those which reduce speed hanges, result in motor-fuel savings, as do nprovements which decrease resistance to ehicular movement and reduce the energy equirements needed for operation.

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[1] This article was presented at the 39th Annual Meeting of the Highway Research Board, Washington, D.C., January 1960. Dr. Claffey is an Associate Professor of Civil Engineering, Catholic University.

A reduction in resistance to movement and he corresponding reduction in fuel use at any ;iven speed results through improvement ,hich effect reduction of surface roughness, eduction of rate of rise and fall, and reducion of curvature. These improvements, howver, frequently permit higher operating peeds which, because of greater air and rollng resistances at higher speeds, result in an ncreased rate of fuel consumption.

The frequencies of stop-and-go and slowown time losses are reduced through the rovision of grade separations and access ontrol to reduce the number of access points nd through construction of additional lanes o relieve congestion. In addition, the freuency of slowdown operations is reduced hen curves sharp enough to require vehicles o reduce speed are removed through alinenent changes.

Savings Used in Benefit Studies

The saving in either fuel or time consumpion due to any one type of highway improvenent is the difference between the amount onsumed if the improvement were made, nd the amount if the improvement were not nade. Where two or more types of improvenent are made at the same location at the ame time, the savings for each can be comuted by assuming that the other improvenent is already completed. For example, if a ighway reconstruction project involving oth upgrading of surface and reduction of ise and fall is considered, the saving in fuel onsumption for operation on the improved urface rather than on the old surface for the ew rate of rise and fall will be the same egardless of the changed rate of rise and fall; nd the saving due to reduction of rise and all on the improved surface is unchanged by he fact that the surface had been improved.

The difference in fuel saving for the condiions before and after an improvement is a rue measure of fuel benefit even when the articular improvement makes possible higher perating speeds which usually increase fuel onsumption. An example is surface improvenent. When a gravel road is improved with a high-type surface, the nominal highway peed will increase. The fuel saving that ighway-users will realize through surface mprovement is the difference between the uel consumption on the old surface at the ominal highway speed for the old road efore improvement, and the fuel consumption at the same speed on the improved surface. The fact that users elect to travel faster on the improved road with a corresponding ncrease in the rate of fuel consumed does not nullify the saving in fuel use, at the lower speed, made possible by the improvement. Any increase in fuel consumption due to the higher operating speed should be considered separately and included in benefit studies as a negative fuel benefit.

The analysis of user benefits for any highway improvement project can be made most satisfactorily by computing separately the savings for each type of improvement involved and then totaling them. Care must be exercised when summing the savings that the same saving is not counted twice. For example, where a two-lane gravel road is reconstructed as a four-lane divided highway with high-type pavement, both upgrading the surface and increasing the number of lanes permit higher operating speeds with the consequent reduction of time consumption, but the time saving for the higher speed can be included only once.

Required data

For the computation of time and fuel savings resulting from a planned improvement, certain data are required. First, the average gross operating weight of each class of vehicle and the number of vehicles within each class expected to use the route can be obtained by traffic studies and loadometer studies. Second, the information on planned physical changes of the road can be obtained from investigation of the site and by study of the improvement plans.

A third requirement, the effect each type of improvement has on speeds, frequency of speed changes, and the length of stops or delays, can be obtained partially from reported studies (*1*)[2] and from investigation of the traffic operations at the improvement site.

The fourth requirement, determination of the reduction of resistances to movement at constant speeds, can be made from published graphs illustrating fuel consumption as affected by rate of rise and fall for vehicles weighing 10,000 pounds or more (*2*) and for passenger cars (*3*). These data are limited to only certain vehicle classes and gross operating weights, however, and are not sufficiently comprehensive for a general benefit analysis.

Test Vehicles and Procedures

The time and fuel consumption study undertaken by the Bureau of Public Roads in 1959 involved three vehicles—a passenger car, a pickup truck, and a 2-axle, 6-tire, dump truck. These three classes of vehicles accounted for over 98 percent of the vehicle-miles traveled in 1956 by all highway vehicles other than buses and the larger commercial vehicle combinations.[3] Such vehicles were being studied at the same time and for the same purpose by the University of Washington, with financial assistance from the Bureau of Public Roads (*4*).

Data for the passenger car were obtained for a single loading condition, while data for the trucks were obtained for various weight loads—enough to cover the lower range of gross vehicle weights for single-unit trucks. The passenger-car load consisted of two persons, the driver and an observer. The pickup truck was operated with no load except for the driver and observer, and with a load approximately equal to full load capacity. The dump truck was operated with no load and with one-half full load.

Table 1.—Data for vehicles used in the test operations [1]

Type of vehicle	Gross vehicle weight (pounds)	Engine power		Cross-sectional area [2] (sq. ft.)
		Net hp.	R.p.m.	
Passenger car	3,850	123	4,200	33
Pickup truck:				
No load	3,860	120	4,000	38
Full load	5,340	120	4,000	38
Dump truck:				
No load	10,200	89	2,800	54
Half load	15,300	89	2,800	54

[1] All vehicles were 2-axle, 6-cylinder vehicles. The passenger car had an automatic transmission; the trucks were equipped with manual transmissions.

[2] Frontal area of vehicle, derived by multiplying overall height by overall width.

The passenger car was a 6-cylinder 1957 standard 4-door sedan with a 3-speed automatic transmission. At the time of the tests it had been in service for 2 years, during which time it had traveled a distance of 30,000 miles. The weight and additional pertinent information for the three vehicles are given in table 1.

The pickup truck used was a 6-cylinder, manual shift 1959 model with a GVW (gross vehicle weight) of 4,900 pounds. The 6-cylinder, 1950 medium-type dump truck had been in service for 50,000 miles. Both trucks were checked on a dynamometer prior to the tests and the efficiency of combustion measured with an exhaust analyzer at a wide range of loads. At the time of the tests the vehicles were operating at near optimum efficiency.

Test run procedures

Data on the time and fuel consumption of these vehicles were obtained from a series of test runs made over a nearly straight section of Virginia route 350 south of Washington, D.C. This is a divided highway with four 12-foot lanes of portland cement concrete, with well-built shoulders of firmly compacted gravel 10 feet wide. The test runs were made between two fixed end points set 8,000 feet

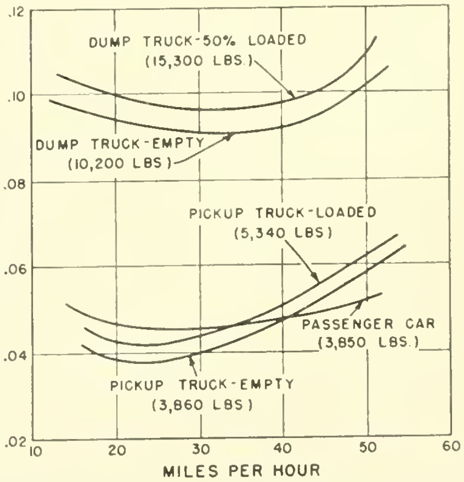

Figure 1.—Fuel-consumption rates at constant speed on a level, straight, concrete surface.

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[2] Italic numbers in parentheses refer to the references on p. 21.

[3] *Traffic and Travel Trends, 1956,* by T. B. Dimmick. PUBLIC ROADS, *vol. 29, No. 11,* Dec. 1957.

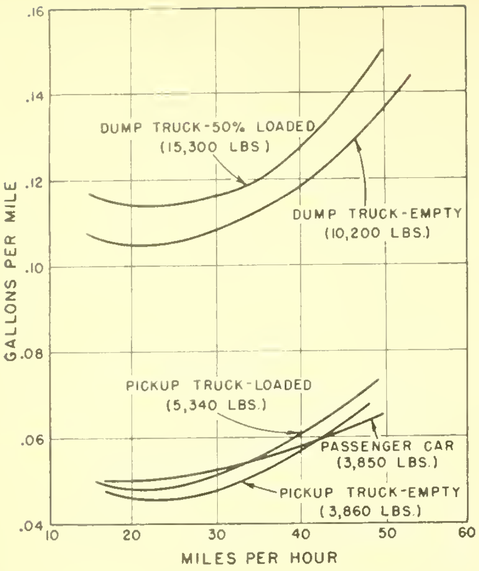

Figure 2.—Fuel-consumption rates at constant speed on a level, straight, gravel surface.

(1.515 miles) apart. These points were at nearly the same elevation and the rate of rise and fall between them was less than 0.2 foot per 100 feet.

The following types of test runs were made between end points of the test section: Constant speed runs on the paved surface at indicated speeds of 15, 25, 35, 45, and 55 miles per hour; constant speed runs on the gravel shoulder at the same speeds; stop-and-go runs on the paved surface at indicated operating speeds the same as for the constant speed runs; and slowdown runs (10 mile per hour speed reduction only) on the paved surface at the same operating speeds. Three runs of each type were made for each vehicle and load in each direction at each of the given speeds. The idling fuel consumption was obtained for each vehicle at engine speeds of 450, 550, 650, and 750 revolutions per minute.

The runs were made by driving the vehicle over the test section, measuring the amount of travel time and fuel consumed between end points and recording the direction of travel, time of day, fuel temperature, and run speed as indicated on the vehicle speedometer. F the constant speed runs no other data we taken. On the stop-and-go and slowdow runs the vehicle was brought to a stop or tl speed reduced by 10 miles per hour ar immediately accelerated back to schedul speed, as many times as possible, passing bot end markers at run speed. Additional dat recorded for these runs were the time durir which acceleration took place after each sto or slowdown, the number of stops and slov downs, and the number of gear changes fo each acceleration. The rates of speed chang used for both the stop-and-go and slowdow operations during deceleration and accelera tion were those of the typical driver unde ordinary conditions (*5*).

Recording the date and time made it poss ble to determine wind direction and velocit at the time of each run by reference to win data collected by the U.S. Weather Bureau a a nearby station.

The first step in the analysis of the field dat was to compute the true speed of each test ru The indicated run speeds, read on the speec

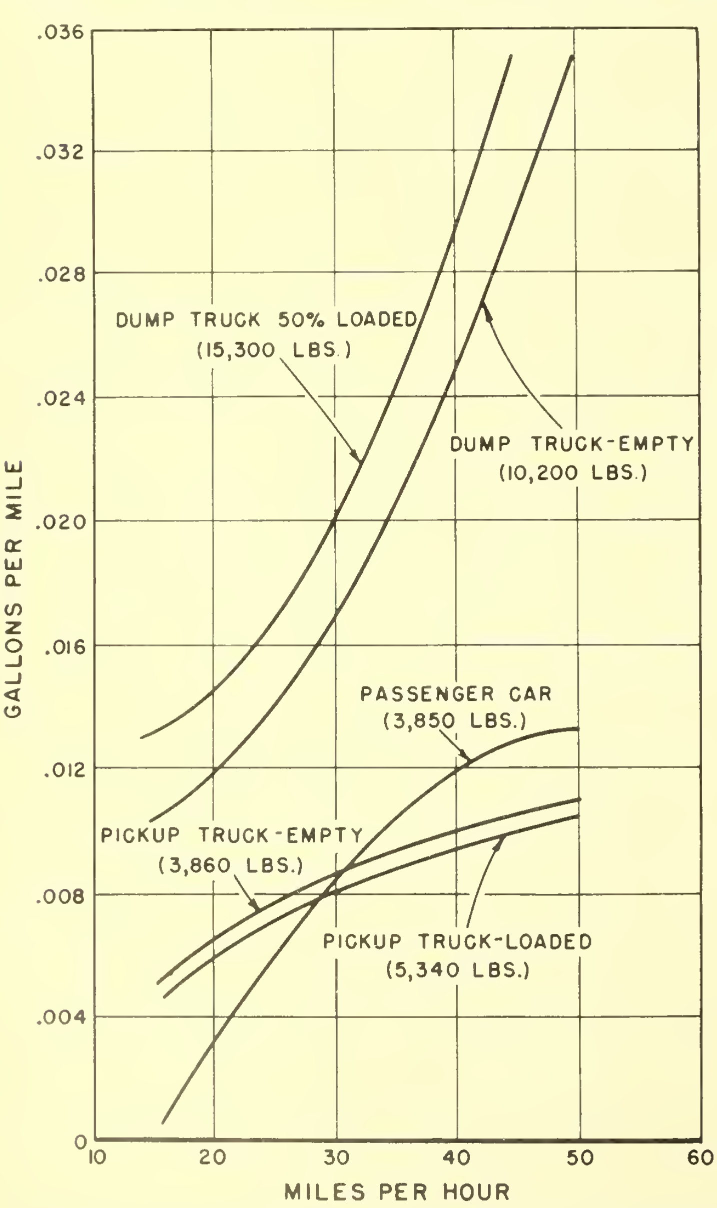

Figure 3.—Savings in fuel consumption at constant speeds effected by improvement from a gravel surface to a concrete surface.

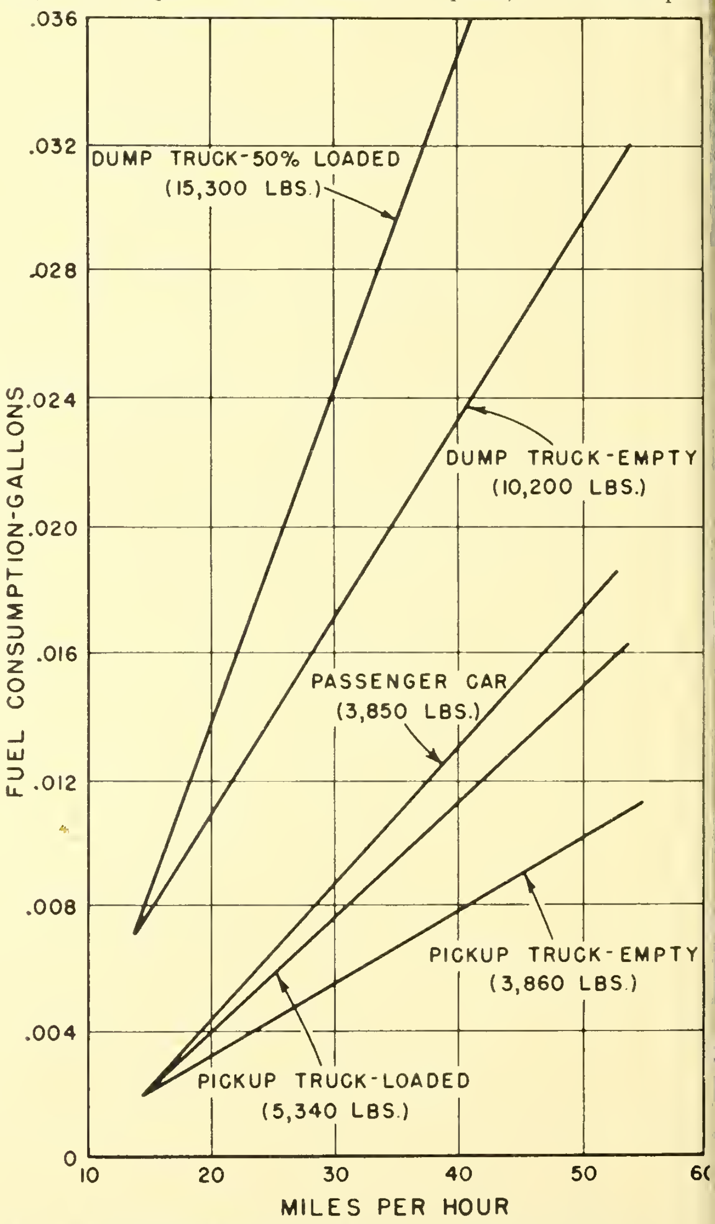

Figure 4.—Excess fuel consumed by a vehicle coming to a stop an then returning immediately to former speed.

meter, were recognizedly inaccurate but were sed during the test because it was easier for ie driver to maintain a given speed consistntly if he had a definite speed to attain rather han attempting to hold the speedometer neele at such a point on the dial that the run peed would be the true speed. The true peed was computed from the known run disance and the run time recorded for the contant speed runs.

Fuel consumption was measured directly nd accurately by noting the amount of fuel rawn out of the reservoir of a burette type uelmeter. Since the volume of fuel measured aried with the temperature, all fuel readings vere corrected to 30° C. (86° F.)—the average uel temperature during the period of the tests.

Application of Test Results

Corrected fuel-consumption values in gallons er mile were computed for each constant peed run on both the paved and gravel suraces. Curves drawn from the averages of hese values for each run speed for each vehile type and weight, plotted against true speed, re shown in figures 1 and 2.

The curves in figures 1 and 2 may be used to stimate the change in rate of fuel consumpion in gallons per mile which will result when ominal highway speeds are increased through ighway improvement on roads carrying raffic volumes somewhat less than their caacity volume. Since nominal highway speed s the average operating speed between points vhere vehicles are slowed or stopped by highvay impedances, the application of these curves is not restricted by the effect of such highway impedances.

In the case of highway improvements where more lanes are to be added to provide greater capacity, when capacity before improvement is less than the 30th hour volume, figures 1 and 2 may be used to estimate the fuel consumption after improvement when vehicle speeds are relatively uniform. However, the lower speeds before improvement are largely due to congestion and include frequent decelerations and accelerations. The rate of fuel consumption before improvement may be estimated by adding to the values found in figures 1 and 2 the amount of additional fuel consumed by slowdowns. The average number of slowdowns may be determined by making suitable speed-delay studies over the route before improvement, and the additional fuel consumption due to a slowdown may be estimated by using the curves in figure 6 (p. 20).

The curves presented in figure 3 show the fuel savings in gallons per mile effected by improvement from a gravel surface to a paved surface. The ordinates of these curves equal the difference between the fuel consumption for operation on a paved surface and operation on a gravel surface at the same speeds.

In using figure 3, the saving due to surface improvement should be read from the curves for the nominal highway speed of the gravel-surfaced road before improvement. A reduction in road roughness, of course, will permit higher speeds and most users take advantage of this even though higher speeds increase fuel consumption for speeds above approximately 35 miles per hour. It is at the discretion of the highway user whether he operates on the improved surface at the same speed as on the rougher surface and saves on fuel use, or operates at a higher speed and pays for the higher speed (and thus time saving) through increased fuel consumption. In either case the saving is made available to the user. The nominal highway speed of modern vehicles on a gravel or loose surfaced road is between 30 and 35 miles per hour; for any particular road under study it should be obtained by making a spot speed study.

Stop-and-go operations

The additional fuel consumed for a stop-and-go operation was found by dividing the number of stop-and-go operations made on each run into the difference between the amount of fuel used for the stop-and-go run and the average amount of fuel used for the constant speed runs on the paved surface at the same run speed. This difference was the additional amount of fuel used by a vehicle to come to a stop and accelerate back to the constant speed. The fuel consumption curves for stop-and-go operations at various speeds are presented in figure 4.

A similar procedure was used to compute the additional time consumption for a stop-and-go operation. Figure 5 shows curves of excess stop-and-go time consumption as a function of true speed. The time and fuel consumption shown in figures 4 and 5 do not include the time or fuel consumed while a vehicle is stopped, but only the additional consumption for the actual stop-and-go maneuver.

Engine idling consumption rates

The idling fuel-consumption rates of the test vehicles are presented in table 2. The data shown were obtained with the vehicle stationary and the engine warm. For the trucks, the rates were obtained in forward gear with the clutch disengaged. The rate for the passenger car equipped with an automatic transmission was measured with the transmission in neutral position and also in drive position with the brakes set. Idling fuel-consumption values are given for four different engine speeds—the average of these being recommended for use in benefit studies.

Figures 4 and 5 and table 2 are particularly useful for estimating the savings which will result if an intersection controlled by traffic signals or stop signs is eliminated through construction of a grade separation structure.

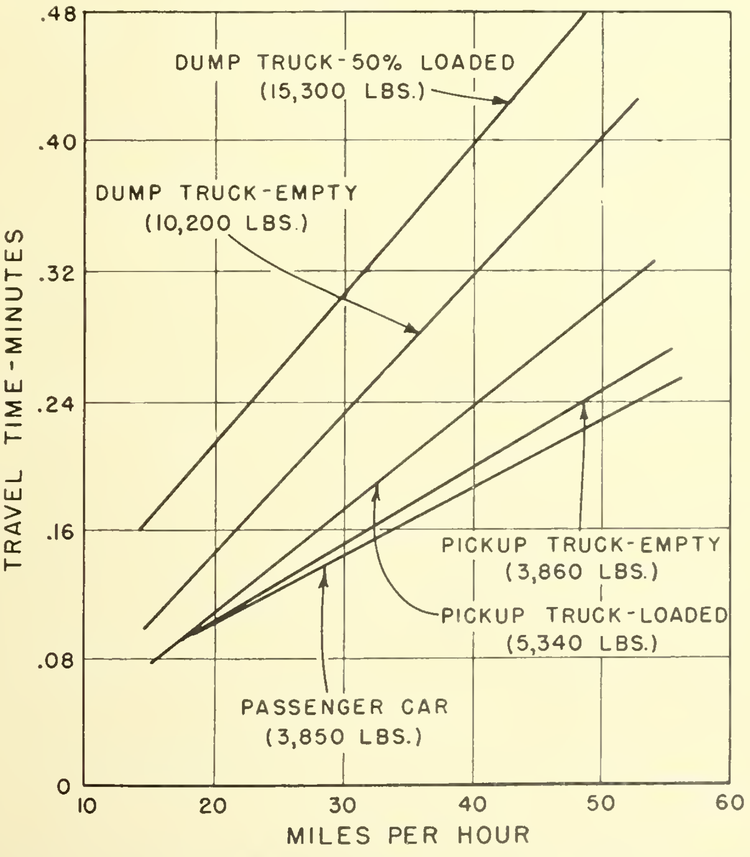

Figure 5.—Excess time consumed by a vehicle coming to a stop and then returning immediately to former speed.

Table 2.—Fuel consumption rates for idling vehicle

Engine revolutions per minute	Fuel consumption in gallons per minute				
	Passenger car			Pick-up truck	Dump truck
	Transmission in neutral	Transmission in drive	Average		
450	0.005	0.005	0.005	0.006	0.009
550	.006	.008	.007	.007	.010
650	.006	.009	.007	----	.011
750	.007	.014	.010	.008	.013
Average	.006	.009	.007	.007	.011

Slowdown operations

The additional fuel and time consumption curves for a slowdown of 10 miles per hour with immediate resumption of speed, at various speeds, are shown in figures 6 and 7. Periods of operation at reduced speed are excluded. The procedures used for computing these values due to slowdowns were similar to those used for computing the fuel and time consumption of stop-and-go operations.

The applicability of the curves in figures 6 and 7 is limited to improvements that eliminate highway impedances which cause vehicles to reduce speed by about 10 miles per hour. This limitation is not serious, however, since most slowdowns of importance in highway-user benefit studies are about 10 miles per hour. Preliminary analysis of data taken from extensive speed-delay studies made in 1958–59 with a passenger car showed that speed reductions of up to 3 miles per hour were a normal part of uniform driving and were not eliminated through highway improvements, and that the average of the speed reductions in excess of 3 miles per hour was about 10 miles per hour. In addition, it was recently established that the average speed reduction of motortrucks when slowed by highway or traffic impedances is 11.4 miles per hour (6).

The curves in figures 6 and 7 may be used to estimate savings in fuel and time consumption through the elimination of a variety of speed reduction factors. In the case of a curve elimination, test runs should be made before improvement to determine the average amount of slowdown caused by the curve. Where crossroads or driveway entrances are to be eliminated, test runs should be made to establish the average percentage of driveways at which through vehicles are forced to reduce speeds because of other vehicles entering or leaving, and the average value of such speed slowdowns.

If the average speed reductions found for curves and driveway entrances are between 8 and 12 miles per hour, figures 6 and 7 may be used to compute fuel and time savings. If the average speed reduction is more or less than 8 to 12 miles per hour, the fuel and time savings may be estimated by assuming that the magnitude of these savings is proportionate to the magnitude of the speed change.

Practical Illustrations

Two examples will illustrate how figures 1–7 and table 2 may be used to compute the fuel and travel-time savings arising from particular improvements.

Example A.—On a 10-mile length of 2-lane road 24 feet wide, the existing gravel surface is to be replaced by a concrete pavement. Average annual daily traffic on the route is 4,000 vehicles per day; 80 percent of the vehicles are passenger cars and 20 percent are 2-axle, single-unit trucks having an average gross vehicle weight of 10,000 pounds. The nominal highway speed on the route before improvement was found to be 35 miles per hour for all vehicles. It is expected that this will be increased to 45 miles per hour after improvement. Compute the average annual

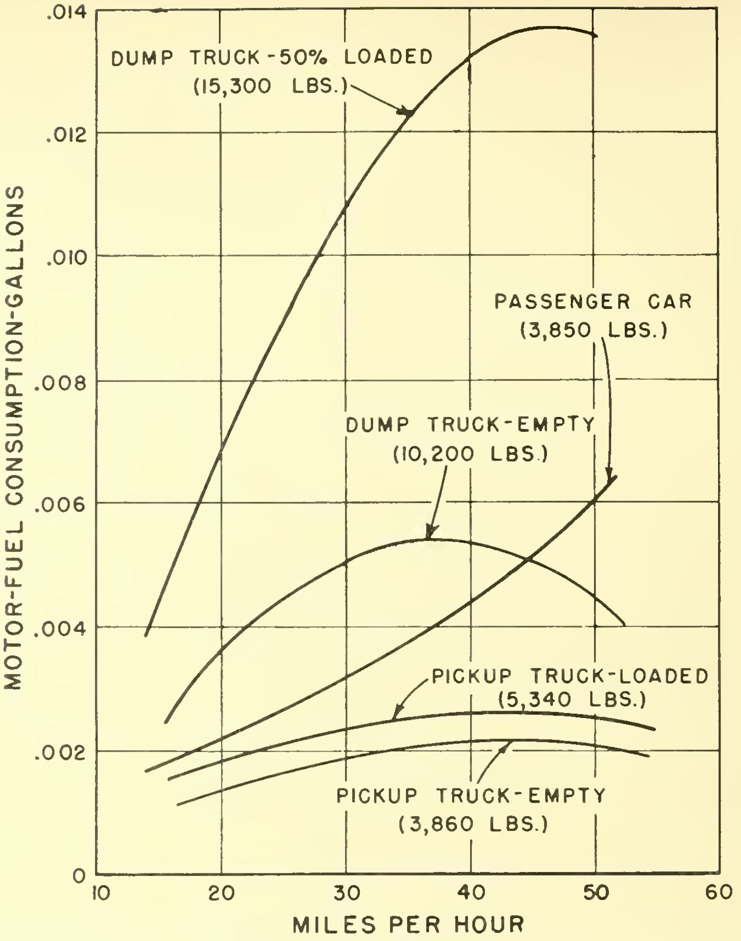

Figure 6.—Excess fuel consumed by a vehicle reducing speed by 10 miles per hour and then returning immediately to former speed.

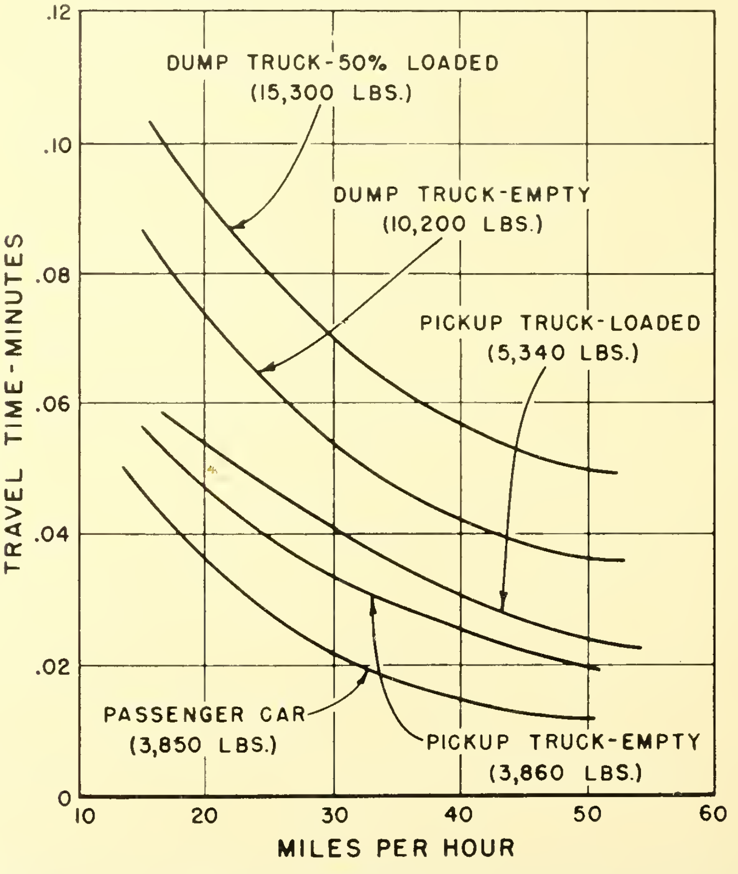

Figure 7.—Excess time consumed by a vehicle reducing speed by 10 miles per hour and then returning immediately to former speed.

ıel savings which may be attributed to this ıprovement.

Solution.—Annual number of vehicles using ıe route:

ıssenger cars	4,000×365×80%=1,168,000
·ucks	4,000×365×20%= 292,000

From figure 3, fuel savings due to surface ıprovement at nominal speed of 35 miles ·r hour are 0.010 gal. per vehicle-mile for ıssenger cars and 0.021 gal. for trucks. rom figure 1, the increases in fuel consumpon due to speed increase from 35 to 45 .p.h. are 0.003 gal. per vehicle-mile for ıssenger cars and 0.004 gal. for trucks.

Annual fuel savings for the 10-mile im·ovement:

ıssenger cars	1,168,000×10×(0.010−0.003)=81,760 gal.
·ucks	292,000×10×(0.021−0.004)=49,640 gal.

The total annual fuel savings thus amount 131,400 gallons. At 30 cents per gallon, ıis represents an annual savings of $39,400.

Example B.—A grade-separation structure planned at the interesection of a 4-lane vided parkway and a 2-lane crossroad where affic signals now control vehicle movements. he average annual daily traffic volumes on ıe 4-lane and 2-lane routes are 20,000 and 000 vehicles per day, respectively. All ·hicles on the parkway are passenger cars;) percent of those on the crossroad are pasnger cars and 20 percent are 2-axle, singleıit trucks having an average gross weight of),000 pounds. The nominal highway speed ı the parkway is 45 miles per hour and on ıe crossroad, 30 miles per hour. Turning ovements at this intersection are so few in ımber that they may be neglected. It was ·termined from a study of traffic movements ıat on both routes traffic signals caused 25 ·rcent of the vehicles to stop, with an aver·ge time loss while idling per stop per vehicle of 20 seconds or 0.33 minute. Compute the annual fuel and time savings which will result from this improvement.

Solution.—Annual number of vehicles stopped on the parkway at the intersection:

Passenger cars	20,000×365×25%=1,825,000

Annual number of vehicles stopped on the crossroad at the intersection:

Passenger cars	4,000×80%×365×25%=292,000
Trucks	4,000×20%×365×25%= 73,000

From figure 4, unit fuel savings per stop-and-go for passenger cars are 0.015 gal. at nominal speed of 45 m.p.h. and 0.009 gal. at 30 m.p.h.; for trucks, 0.017 gal. at 30 m.p.h. From figure 5, unit time savings per stop-and-go for passenger cars are 0.21 min. at nominal speed of 45 m.p.h. and 0.14 min. at 30 m.p.h.; for trucks, 0.23 min. at 30 m.p.h. From table 2, fuel consumption while idling for passenger cars is 0.007 gal. per min.; for trucks, 0.011 gal. per min.

Annual fuel savings for passenger cars on the parkway:

Stop-and-go	1,825,000×0.015=27,375 gal
Idling	1,825,000×0.33×0.007= 4,216 gal.

Annual fuel savings for passenger cars on the crossroad:

Stop-and-go	292,000×0.009=2,628 gal.
Idling	292,000×0.33×0.007= 674 gal.

Annual fuel savings for trucks on the crossroad:

Stop-and-go	73,000×0.017=1,241 gal.
Idling	73,000×0.33×0.011= 265 gal.

The total annual fuel savings thus amount to 36,399 gallons. At 30 cents per gallon, this represents an annual savings of $10,900.

Annual time savings for passenger cars on the parkway:

Stop-and-go	1,825,000×0.21=383,250 min.
Idling	1,825,000×0.33=602,250 min.

Annual time savings for passenger cars on the crossroad:

Stop-and-go	292,000×0.14=40,880 min.
Idling	292,000×0.33=96,360 min.

Annual time savings for trucks on the crossroad:

Stop-and-go	73,000×0.23=16,790 min.
Idling	73,000×0.33=24,090 min.

The total annual time savings thus amount to 1,163,620 minutes. Taking, for the sake of example, a value of 1½ cents per minute, the annual time savings are worth $17,400. Thus the total annual fuel and time savings occasioned by construction of the grade-separation structure can be valued at $28,300.

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Fuel and time consumption rates for trucks in freight service. See pp. 22–31 of this issue of Public Roads.

Fuel and Time Consumption Rates for Trucks in Freight Service

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported [1] by MALCOLM F. KENT
Transportation Economis

The number of times a truck must change its speed in a mile of travel increases with the density of traffic, according to an analysis of data derived from studies conducted in 1957 and 1958 of rural and urban travel in five States—data necessary in the analysis of highway-user benefits.

Using a congestion index, which indicates that speed changes per mile increase uniformly with average daily traffic for different types of highway, together with the rates of fuel and travel time consumed during a change in vehicle speed, the added cost of operating at nonuniform speed could be assessed.

This article also shows that, for the gross vehicle weights observed, smaller and less powerful engines give better fuel economy, but their use carries a penalty of increased time-consumption (lower road speeds) at the higher gross vehicle weights. Trucks with diesel engines were found to travel about 50 percent more miles on a gallon of fuel than trucks with gasoline engines of approximately equivalent power and gross weight characteristics.

ONE of the greatest voids in the data available for the analysis of highway-user benefits accruing through the improvement of highway facilities has been reliable fuel- and and time-consumption rates of commercial motor vehicles operating in actual service. To help fill this void the Bureau of Public Roads developed a program for obtaining this information. Ohio State University, the Universities of Michigan and Washington, and a transportation consultant from the University of Maryland were engaged to measure fuel consumption and overall travel time of selected trucks in rural and urban line-haul service and in city pickup and delivery service, under traffic conditions ranging from restricted to free flowing. This study group obtained the cooperation of private, government-owned, and for-hire highway freight carriers. Three of the studies were conducted simultaneously during the summer of 1957, and one during the summer of 1958.

A principal concern of highway planners of a few decades ago was the surfacing of dirt roads. Today, a principal concern is the elimination of frictional factors that impede the free flow of traffic on paved roads. Eliminating stops occasioned by stop signs and traffic lights, the widening of pavements or the adding of more lanes, the designing of highways with easier grades and curves, and the upgrading of other features that cause reduction in normal driving speeds, are factors that are now of primary importance.

In addition to improving the safety and efficiency of traffic flow, such improvements result in direct benefits to road users. Savings in motor fuel and time costs are two of the principal benefits that result, and they are directly affected by the elimination of frictional factors that impede the free flow of traffic. The over-all purpose of the studies described in this report was to provide data on fuel consumption and travel time for various vehicle types and traffic conditions, which could be used in the economic analyses of road-user benefits.

Summary of Findings

Major findings of the studies are summarized in the following paragraphs.

1. The fuel consumption in gallons per mile of motor trucks operating in rural and urban line-haul service increased with the power of the engine for equivalent gross vehicle weights.

2. Operating over identical rural line-haul routes, diesel-powered trucks were found to travel about 50 percent more miles on a gallon of fuel than gasoline-powered trucks of approximately equivalent power and gross vehicle weight. In terms of fuel consumption, this means that diesel-powered trucks consumed about 66 percent of the gallonage used by gasoline-powered trucks.

3. The consumption of gasoline per mile by trucks was 25 to 30 percent higher in urban areas than in rural areas.

4. The average truck speeds, including all stops and slowdowns, were found to be 37 miles per hour in rural line-haul operation, 19 miles per hour in urban line-haul operation, and 11 miles per hour in city pickup and delivery. For free-flowing traffic, the comparative speed for trucks in rural line-haul operation was 40 miles per hour.

5. The usefulness of speed changes per mile as a congestion index was demonstrated by proving that speed changes per mile increased uniformly with average daily traffic for different types of highways. Knowing the numbe of speed changes saved, the proportion of stop and slowdowns, and the magnitude of each it is possible to use this index to compute th added cost of fuel and time caused by spee changes, when the extra fuel and tim consumed during a speed change is known.

6. The stops on rural highways, made fro the average truck speed, represented 11 per cent of all deviations from desired speeds whereas the stops on urban streets represente 45 percent of all deviations from desire speeds.

7. The average number of speed change per mile was found to be 1.66 for rural lin haul, 4.97 for urban line-haul, and 6.91 fo city pickup and delivery operations.

Definition of Terms

To avoid misinterpretation of the result certain terms used in this article are define

Fuel consumption.—Gallons of gasoline diesel fuel consumed per mile of highwa travel. The conversion from gallons per mi to miles per gallon can easily be made sinc one is the reciprocal of the other.

Travel time.—Minutes required to trav 1 mile. Minutes per mile can be convert to miles per hour by dividing 60 by t minutes per mile.

Stop.—Bringing a motor vehicle to complete stop.

Slowdown.—A reduction in speed of a mot vehicle of more than 3 miles per hour witho coming to a stop.

Speed change.—All motor vehicle acceler tions and decelerations effecting a spe change of more than 3 miles per hour, includi both stops and slowdowns.

Average gross vehicle weight.—The avera of the individual gross vehicle weights several vehicles, all falling within the san class interval of gross vehicle weight.

Engine cubic-inch displacement.—The cros sectional area of a cylinder multiplied by t length of piston stroke, which gives t cylinder displacement; multiplied by t number of cylinders.

Net horsepower.—The brake horsepower the engine, operating with all its norm accessories, that is available at the clutch its equivalent. It is the gross horsepow minus the horsepower absorbed by fa compressor, generator, etc. For all practic

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[1] This article was presented at the 39th annual meeting of the Highway Research Board, Washington, D.C., January 1960.

Table 1.—Route termini, route numbers, distances, and rates of rise and fall of rural highways traveled by observed line-haul trucks

Termini		Numbered routes	Mileage [1]	Rate of rise and fall [2]
From	To			
Washington, D.C.	Baltimore, Md.	Md. 193, U.S. 1	32.6	1.58
Do	Richmond, Va.	Va. 350, U.S. 1	95.5	1.42
Columbus, Ohio	Cleveland, Ohio	Ohio 3, 61, U.S. 42	128.4	1.41
Do	Parkersburg, W. Va.	U.S. 33, 50	108.8	.63
Do	Wheeling, W. Va.	U.S. 40	119.9	1.70
Detroit, Mich.	Lansing, Mich.	U.S. 16	80.5	.59
Do	Toledo, Ohio	U.S. 25	55.4	.16
Do	Three Rivers, Mich.	U.S. 112, 12, Mich. 60	151.7	.48
Seattle, Wash.	Aberdeen, Wash.	U.S. 99, 410	95.5	1.25
Do	Bellingham, Wash.	U.S. 99	75.5	1.28
Do	Centralia, Wash.	U.S. 99	74.7	1.09
Do	Chehalis, Wash.	U.S. 99	80.7	1.09
Do	Everett, Wash.	U.S. 99	18.5	1.87
Do	Longview, Wash.	U.S. 99, 830	120.7	.95
Do	Mt. Vernon, Wash.	U.S. 99	53.9	1.24
Do	Olympia, Wash.	U.S. 99	53.1	1.29
Do	Portland, Oreg.	U.S. 99	161.2	.93
Do	Tacoma, Wash.	U.S. 99	23.9	1.59
Do	Yakima, Wash.	U.S. 10, 97	139.1	1.35

[1] Between municipal boundaries of terminal cities.
[2] In feet per 100 feet of distance.

rposes, net horsepower is assumed to be 90 rcent of the gross horsepower.

Total rise and fall.—The arithmetic sum of e vertical rise and fall in feet for any section highway. The rise in one direction of ivel will become the fall in the opposite rection. The total rise and fall is the same gardless of the direction of travel.

Rate of rise and fall.—The total rise and fall any section of highway in feet divided by e length of section in hundreds of feet. It not to be confused with the percent of grade. is equivalent to the average percent of ide only when either the rise or fall is 100 rcent of the total rise and fall.

Description of Test Routes

The four studies were conducted in the neral areas of Maryland-District of Colum-a-Virginia, Ohio, Michigan, and Washington. ie line-haul (intercity) routes with their gins, destinations, route numbers, mileages, d rates of rise and fall are shown in table 1. ie urban extensions of the line-haul routes Cleveland and Columbus, Ohio, Detroit, ich., Baltimore, Md., Washington, D.C., attle, Wash., and some smaller munici-lities were studied separately from rural e-haul operation. These generally followed numbered routes until diversion was cessary to reach the trucking terminal or livery warehouse.

City pickup and delivery service was idied in Detroit, Columbus, Seattle, and ashington, D.C. All such operations were irregular routes except for the postal livery service trucks which followed the ne routes each day to the various substations Columbus. The types of service varied m large tractor-truck semitrailer combi-tions delivering grocery products from rehouses to retail stores and motor fuel m wholesale storage tanks to retail filling tions, to panel and van-type trucks engaged package or linen delivery service. Rise d fall rates were estimated for Columbus, troit, and Washington, D.C., at approxi-tely 0.5 foot per 100 feet. Rates of rise d fall for routes were recorded for Seattle, and ranged from 1.9 to 2.3 feet per 100 feet. However, the variations in rates of rise and fall among routes were not of sufficient magnitude to cause significant changes in fuel and time consumption.

Description of Test Vehicles

The gasoline- and diesel-powered tractor-truck semitrailer combinations, made available by commercial carriers for line-haul observation, are described in table 2 according to type, engine displacement, and net brake horsepower. City pickup and delivery gasoline-powered vehicles, consisting of panel and other single-unit trucks and tractor-truck semitrailer van and tank combinations, are similarly described in table 2.

Where the size and weight restrictions of the particular State permitted, three vehicles were observed in each State within each of the following weight groups:

Rural and urban line-haul (pounds)	*City pickup and delivery (pounds)*
20,000–29,999	5,000– 9,999
30,000–39,999	10,000–19,999
40,000–49,999	20,000–29,999
50,000–59,999	Over 30,000
60,000–69,999	

Test Procedures

After receiving permission from fleet owners to use their vehicles for test purposes, in the course of their normal runs, a fuel meter was placed in the cab of each gasoline-powered truck and connected to the fuel lines of the engine between the tank and the carburetor. The fuel meter could be read by a person sitting next to the driver. The fuel tank was filled at the start of each trip and was filled again at the end of the trip; any fuel added enroute was, of course, recorded. This overall record of fuel consumption was used to check the accuracy of the meters.

Diesel-engine trucks, in which excess fuel is recirculated from the engine to the fuel tank, required a different type of meter installation. To circumvent the multimetering of the same fuel, a small-volume, constant-level tank was installed in the fuel line between the engine and the main fuel supply tank. The engine fuel pump drew only from this feed tank, to which all excess recirculated fuel was returned. Fuel consumed by the engine was drawn from the feed tank, and a constant level was maintained in the feed tank through a float arrangement and an auxiliary fuel pump supplying additional fuel from the main supply tank through a fuel meter unit. In this manner, the fuel meter recorded only the actual quantity of fuel consumed by the engine.

Table 2.—Characteristics of commercial motor vehicles made available for study

Number of vehicles	Number of axles and body types [1]	Engine cu. in. displacement	Net brake h.p. of engine [2]	Engine r.p.m.
Line-haul, gasoline:				
1	3–S2–2 van.	302	172	3,600
3	3–S2 van	331	128	3,200
1	2–S2 van	377	126	2,800
12	2–S1 van	386	130	2,800
8	3–S1 van	406	156	2,750
4	3–S2 van	450	146	2,600
2	3–S2 van	461	197	3,200
3	2–S2 van	501	165	2,800
1	3–2 van	531	178	2,880
1	3–S2 van	549	230	3,200
4	3–S2 van	590	225	2,800
Line-haul, diesel:				
5	3–S2 van	743	200	2,100
City pickup and delivery, gasoline:				
2	2 panel	214	73	3,200
1	do	223	126	4,000
1	do	235	123	4,000
1	2 van	220	89	2,800
5	do	228	90	3,000
5	do	248	115	3,400
1	do	260	90	2,500
1	do	261	135	4,000
1	do	263	105	3,400
3	do	271	114	2,800
2	do	272	167	4,400
1	do	282	103	3,200
2	do	320	103	3,000
1	do	386	163	3,000
2	2–S1 van	372	139	3,200
2	do	386	145	3,000
3	do	406	175	3,200
1	2–S2 van	383	150	2,800
2	do	450	150	2,800
1	do	505	175	2,800
2	2–S2 tank	464	170	2,800

[1] Each digit indicates the number of axles of a vehicle or of a unit of a vehicle combination. A single digit, or the first digit of a group symbol, represents a single-unit truck or, if followed by an S, represents a truck-tractor. The S designation represents a semitrailer. A digit, without an S preceding it, in the second or third position of a group symbol represents a full trailer.
[2] Average 140 horsepower for engine sizes 302–406 cu. in., average 171 horsepower for sizes 450–549.

Before the beginning of the test runs each route to be observed was inventoried to locate control points with relation to major changes in traffic flow and to record mileage between control points, rise and fall (through use of an aneroid barometer), number of traffic signs and signals, and number of lanes. Before the start of each run, the observer recorded the vehicle chassis model and year, unladen weight, payload weight, and gross vehicle weight, engine model size and cubic inches of cylinder displacement, and reported net brake horsepower. The weather and condition of the road were also recorded.

The observer, riding in the cab, recorded on each run the following information as he passed the control points: time of day (hour and minute), fuel meter reading (hundredths of a gallon), and odometer reading (tenths of a mile). The magnitude of each speed change of plus or minus 3 miles per hour or more within each section was recorded during the trip. Trips were made at all hours of the day and night, with no change from normal operations being made on account of the study. Drivers were not to change their normal driving habits, and drove at speeds representative of other traffic.

Analysis Procedures

When the fieldwork had been completed, the first step in the analysis procedure was to list the consumption of fuel, travel time, and mileages traveled on each section for each trip, segregating rural from urban data. Speed changes were similarly listed for each section and trip, with stops being shown separately from slowdowns in the Ohio and Washington data. Gallons per mile, minutes per mile, and speed changes per mile were computed separately for line-haul rural trips, for line-haul urban trips, and for city pickup and delivery trips.

Rate of rise and fall

Rise and fall was considered a variable with respect to fuel consumption rates and travel time. No significant variations were found, however, in either parameter for the rather narrow range of rates of rise and fall studied. As shown in table 1, rates of rise and fall for the rural highways studied ranged from 0.16 for the route between Detroit and Toledo, to 1.87 for the route between Seattle and Everett. Of the total mileage studied, 40.6 percent had a rate of rise and fall below 1.0, 47.7 percent had rates from 1.0 to 1.5, and 11.7 percent had rates from 1.51 to 1.87. The average rate of rise and fall for all rural sections studied was 1.22. The results reported for this study reflect the average values for all highwa sections without regard to variations in ris and fall.

Table 3.—Number of trips and total miles observed for gasoline- and diesel-powered motor vehicles, classified by gross vehicle weights

Weight classes (pounds)	Average gross vehicle weight	Gasoline-powered vehicles		Diesel-powered veh.cles	
		Number of trips	Total miles observed	Number of trips	Total miles observed
Line-haul vehicles:					
17,000-18,999	17,000	15	1,111	------	------
19,000-23,999	21,300	55	3,085	------	------
24,000-29,999	27,000	25	2,398	1	60
30,000-37,999	34,500	123	11,740	6	545
38,000-47,999	42,000	98	8,906	12	1,641
48,000-53,999	51,200	64	5,381	8	668
54,000-61,999	59,500	42	3,520	9	1,125
62,000 and over	67,900	31	2,111	12	1,503
Total	------	453	38,252	48	5,542
City pickup and delivery vehicles:					
4,400-4,999	4,600	13	231	------	------
5,000-8,999	6,000	25	1,172	------	------
9,000-12,999	10,500	89	1,775	------	------
13,000-16,999	14,500	51	603	------	------
17,000-20,999	18,500	6	67	------	------
21,000-24,999	22,500	1	33	------	------
25,000-30,499	27,500	80	480	------	------
30,500-36,999	33,300	18	232	------	------
37,000-39,999	38,500	3	81	------	------
40,000-45,999	42,100	5	171	------	------
51,000-51,999	51,300	3	154	------	------
54,000-59,999	57,000	40	64	------	------
62,000-69,999	66,000	32	70	------	------
Total	------	366	5,133	------	------

Table 4.—Summary of fuel-consumption rates for line-haul trucks operating over rural highway, classified by gross vehicle weights and engine power characteristics [1]

Gross vehicle weight (pounds)	Fuel-consumption rates in gallons per mile for vehicles with engine displacement of—							
	302–406 cu. in. gasoline-powered (140 hp.)		450–549 cu. in. gasoline-powered (171 hp.)		590 cu. in. gasoline-powered (225 hp.)		743 cu. in. diesel-powered (200 hp.)	
	Actual rate	Computed rate [2]	Actual rate	Computed rate [2]	Actual rate	Computed rate [2]	Actual rate	Computed rate [2]
17,000	0.150	0.154	0.152	0.173	------	--------	------	--------
21,300	.163	.163	.189	.185	------	--------	------	--------
27,000	.170	.175	.210	.200	0.243	0.241	0.146	0.153
34,500	.196	.191	.229	.219	.247	.253	.176	.162
42,000	.214	.207	.246	.239	.278	.266	.176	.171
51,200	.233	.226	.256	.263	.273	.280	.164	.182
59,500	.233	.244	.289	.285	.287	.294	.189	.193
67,900	------	--------	.298	.307	.314	.307	.212	.203

[1] Average rate of rise and fall, 1.2 feet per 100 feet.

[2] Computed rates are based on the following formulas: 302–406 cu. in., 0.1177+0.00212W; 450–549 cu. in., 0.1288+0.00262W; 590 cu. in., 0.1975+0.00162W; and 743 cu. in., 0.1194+0.001229W. (W=GVW in thousands of pounds.)

Vehicle weight groupings

It was not possible to set up a precis schedule of vehicles and gross vehicle weight to be observed, since the demand for commercial freight in normal operations did not permit the selection of a specified gross vehicl weight. It was hoped that the plan to observe a minimum of three vehicles for each o several weight-class intervals would result i an even distribution within the class interval This, however, was not the case and it wa necessary to form new gross vehicle weigh groupings in the analyses. The most significant groupings for the line-haul and picku and delivery vehicles, together with th number of trips and total miles observed i each grouping, are shown in table 3. It i evident that sizable mileages were logged i each type of service and that a reliable bas exists for the development of fuel consumption and travel time rates.

Engine size groupings

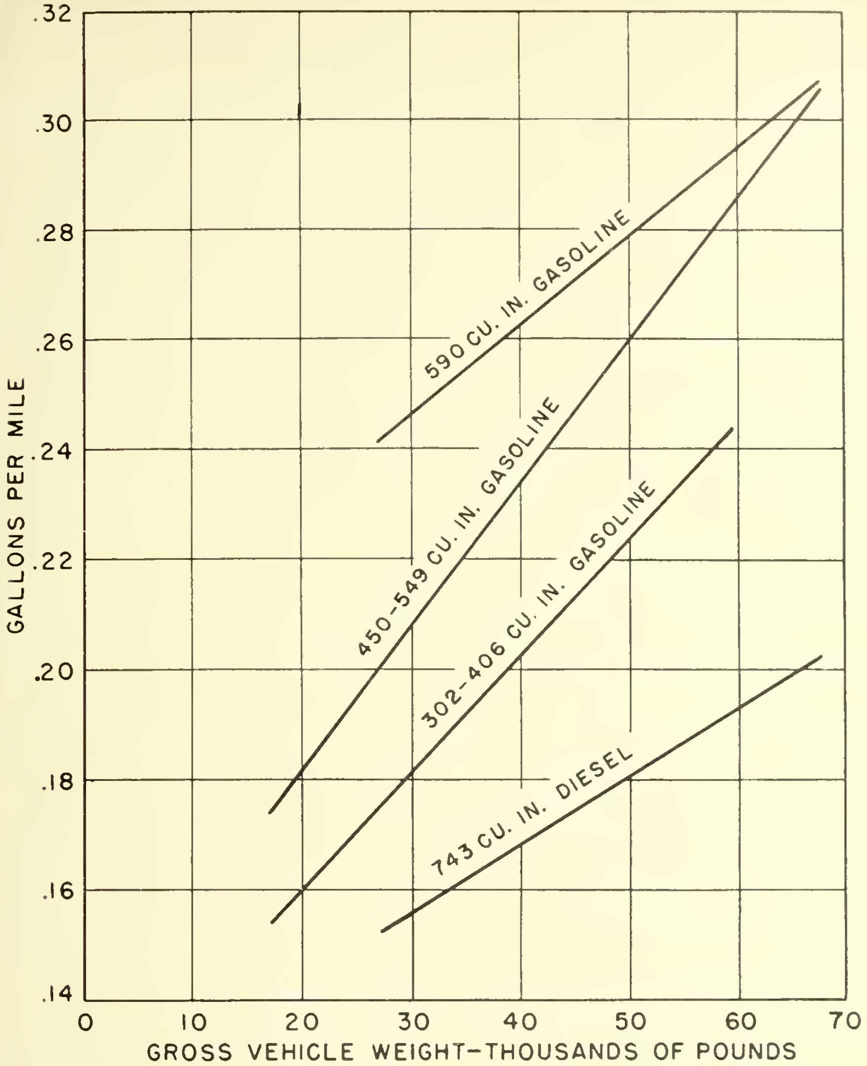
The gasoline-powered vehicles observed o line-haul operations were grouped, for pu poses of analyses, into three engine displacement size groups consisting of 302–406 cubi inches, 450–549 cubic inches, and 590 cubi inches. Vehicles with 743-cubic-inch displacement diesel engines were also studie as a group. The net horsepower for th four groups of engine displacement wer determined to be 140 horsepower for th 302–406-cubic-inch size group, 171 horsepowe for the 450–549-cubic-inch size group, 22 horsepower for the 590-cubic-inch size grou and 200 for the 743-cubic-inch diesel engin

A grouping of city pickup and deliver vehicles by power characteristics was co sidered but found impractical for the purpose of analysis because of the irregularity of th service, which resulted in wide variations i the speed of operation, number of deliverie stops per mile, idling time, and the rate discharge of cargo.

Average Fuel Consumption Rates

A summary of the average rates of fu consumption is shown in table 4. Two fu consumption values are shown for each grou of vehicles with similar power characteristic One is the actual rate and the other is t computed rate, shown in figure 1 as straig line relationships, which were derived fro the actual average values. The rates of ri and fall were 1.18 feet per 100 feet for t 302–406-cubic-inch group, 1.20 feet per 1 feet for the 450–549-cubic-inch group, 1. feet per 100 feet for the 590-cubic-inch grou and 1.22 feet per 100 feet for the 743-cubi inch diesel engine. The variation in rise a fall appeared to be rather insignificant a therefore a valid comparison of the motor-fu consumption rates for the several groupin of vehicles is practical.

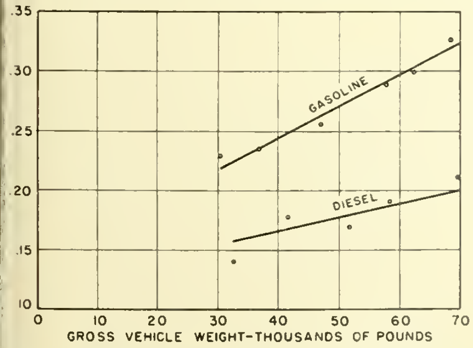
It may be noted that the vehicles with t larger power plants used appreciably mo gasoline for a given average weight. F



igure 1.—Motor-fuel consumption rates of rural line-haul trucks by size of engine for 1.2 rate of rise and fall.

nstance, in figure 1 it is seen that gasoline-owered vehicles in the lowest engine power roup with an average GVW (gross vehicle eight) of 40,000 pounds had a fuel-consumption rate of 0.202 gallon per mile. This ompares with 0.233 gallon per mile for ehicles in the medium power group, which epresents a 15-percent increase; and with .262 gallon per mile for vehicles in the argest gasoline-engine power group, a 30-ercent increase.

Also, it may be seen that the fuel-consumption rate increased with gross vehicle weight. In the medium power group, for instance, a vehicle weighing 20,000 pounds consumed approximately 0.181 gallon per mile, while a vehicle weighing 60,000 pounds consumed 0.285 gallon per mile. However, despite the fuel-consumption rate increase with gross vehicle weight increase, there was a decrease in the fuel consumption per 10,000 pounds of gross vehicle weight. For example, in the medium power group a 20,000-pound vehicle consumed 0.181 gallon per mile, or 0.091 gallon per mile per 10,000 pounds, while a vehicle weighing 60,000 pounds which consumed 0.285 gallon per mile actually consumed only 0.048 gallon per mile per 10,000 pounds, indicating that as gross vehicle weight is increased the fuel economy per unit of gross weight is improved.



igure 2.—Comparison of gasoline and diesel fuel consumption rates of rural line-haul trucks operating over the same routes.

Gasoline and diesel fuel comparison

For the same gross vehicle weight averages, the diesel-powered vehicles consumed considerably less fuel than the gasoline-powered vehicles with approximately the same power characteristics. For example, a vehicle with a 590-cubic-inch gasoline engine and an average GVW of 60,000 pounds consumed approximately 0.294 gallon per mile, while a vehicle with a 743-cubic-inch diesel engine and a similar weight consumed 0.193 gallon per mile. In this case the diesel consumption rate was 66 percent of the gasoline consumption rate. However, the foregoing comparison does not represent results obtained over identical routes.

A comparison of gasoline and diesel fuel consumption rates for vehicles traveling over identical routes was possible from the data obtained in the State of Washington. The diesel-powered combination units traveled a total of 5,542 miles on 48 trips. Twenty-eight of these trips, totaling a distance of 3,617 miles, were traveled over the same routes used by gasoline-powered trucks on 32 trips, totaling 3,966 miles. By grouping gross vehicle weights into class intervals, it was possible to obtain average consumption values that were directly comparable with respect to rise and fall rates and gross vehicle weight. Of the vehicles with gasoline engines, 21 trips were made by vehicles with engines of 461-cubic-inch displacement, 3 with engines of 450-cubic-inch displacement, and 8 with engines of 590-cubic-inch displacement. For the 32 trips, the average net horsepower of the vehicles with gasoline engines was 199 horsepower, as compared with the 200-horsepower diesel engines. The results are summarized in table 5 and the relationships derived from the average rates of fuel consumption are shown in figure 2.

Table 5.—Gasoline and diesel fuel consumption rates for line-haul trucks traveling over the same rural routes, classified by gross vehicle weights [1]

Gross vehicle weight (pounds)	Number of trips	Total miles traveled	Total gallons consumed	Actual rate, miles per gallon	Computed [2] rate, miles per gallon	Actual rate, gallons per mile	Computed [3] rate, gallons per mile
Gasoline:							
30,400	1	63	14. 34	4. 393	4. 452	0. 228	0. 221
36,800	2	284	66. 77	4. 253	4. 224	. 235	. 237
46,800	7	993	254. 89	3. 896	3. 867	. 257	. 263
57,900	14	1, 831	529. 23	3. 460	3. 472	. 289	. 292
62,500	1	142	42. 45	3. 345	3. 308	. 299	. 303
68,300	7	653	213. 25	3. 062	3. 101	. 327	. 318
Total or average	32	3, 966	1, 120. 93	3. 538	--------	. 283	--------
Diesel:							
32,600	1	65	9. 15	7. 104	6. 723	0. 141	0. 158
41,500	7	923	163. 89	5. 632	6. 229	. 178	. 168
51,600	6	618	105. 17	5. 876	5. 668	. 170	. 179
58,100	8	1, 146	219. 87	5. 212	5. 306	. 192	. 187
69,900	6	865	182. 15	4. 749	4. 651	. 211	. 200
Total or average	28	3, 617	680. 23	5. 317	--------	. 188	--------

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[1] Average rate of rise and fall, 1.17 feet per 100 feet.

[2] Computed miles-per-gallon rates are based on the following formulas: gasoline, $5.53486-0.03563W$; diesel, $8.5345-0.05556W$.

[3] Computed gallons-per-mile rates are based on the following formulas: gasoline, $0.14217+0.00258W$ diesel, $0.12106+0.00113W$. (W=GVW in thousands of pounds.)

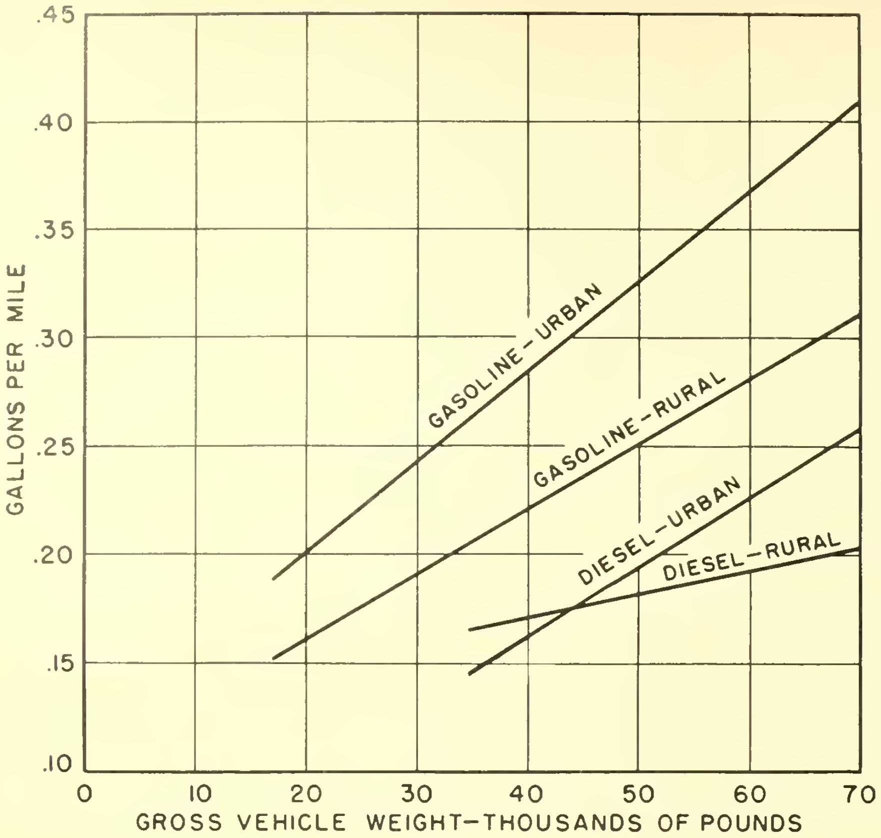

Figure 3.—Comparison of rural and urban motor-fuel consumption rates of line-haul trucks.

For a GVW of 70,000 pounds, as shown in figure 2, the gasoline consumption rate was 0.322 gallon per mile, or 3.11 miles per gallon; and the diesel consumption rate was 0.200 gallon per mile, or 5.00 miles per gallon. In effect the diesel-powered vehicles traveled about 53 percent more miles per gallon of fuel than did the gasoline-powered vehicles. A similar comparison for a GVW of 50,000 pounds indicated that the diesel-powered vehicles traveled about 52 percent more miles per gallon of fuel than gasoline-powered vehicles. A comparison of the average rate for all gasoline-powered vehicles for all 32 trips with that for all diesel-powered trips shows that 51 percent more mileage was obtained by diesel-powered vehicles on the same gallonage of fuel. This relative value is based on the total miles traveled and total gallons consumed shown in table 5. The average diesel consumption rate of 0.188 gallon per mile was 66 percent of the average gasoline consumption rate of 0.283 (also shown in fig. 1).

Rural and urban comparison

The fuel consumption rates for all gasoline- and diesel-powered trucks observed in line-haul rural and urban travel are shown in table 6. The computed rates, obtained from the straight-line relationships shown in figure 3, were derived from the average actual rates. Referring to figure 3, the fuel consumption rates for gasoline-powered vehicles in urban travel appear to be considerably greater than the gasoline consumption rates in rural travel. The fuel consumption percentage differences in rural and urban travel range from a 25-percent difference for a GVW of 20,000 pounds to a 32-percent difference for a GVW of 70,000 pounds.

A comparison of the rural and urban fuel consumption rates for diesel-powered trucks observed in line-haul service, however, shows that there was little percentage difference where the GVW was from 40,000 to 50,000 pounds, but where the GVW approached 70,000 pounds there was a 27-percent higher consumption rate in urban travel.

Again, figure 3 shows the fuel consumption advantage of the diesel engine.

City pickup and delivery vehicles

City pickup and delivery motor-vehicle gasoline consumption rates are shown in figure 4 for two different rates of rise and fall. The straight-line values were derived from actual average values. In Seattle, where the rate of rise and fall averaged 2.1 feet per 100 feet, the gasoline consumption was 18 percent higher at 10,000 pounds GVW and 14 percent higher at 40,000 pounds GVW than the consumption rate in the other three cities where the rise and fall was about 0.5 feet per 100 feet. It will be noted that gasoline consumption increased as gross vehicle weights increased as was the case for line-haul operation. It may also be noted that the consumption rates approximate very closely the values shown in figure 3 for gasoline-powered vehicles in urban line-haul service. Consumption rates for

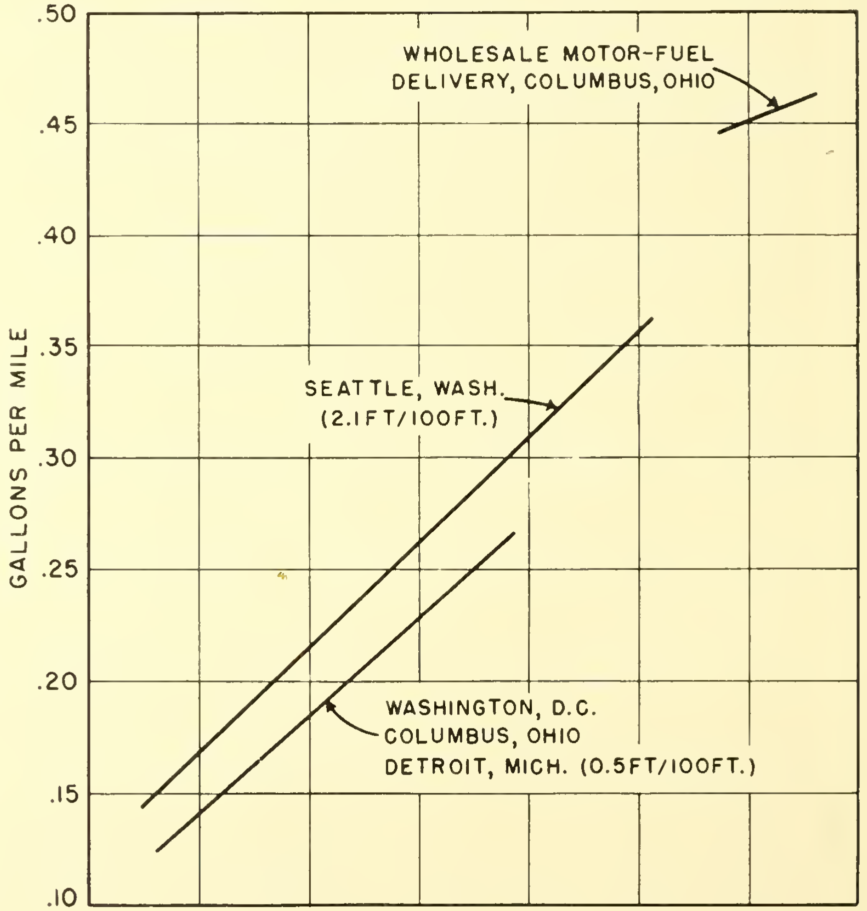

Figure 4.—Motor-fuel consumption rates at different rates of rise and fall for city delivery vehicles.

Table 6.—Gasoline and diesel fuel consumption rates for rural and urban line-haul operations, classified by gross vehicle weights

Gross vehicle weight (pounds)	Gasoline and diesel fuel consumption rates in gallons per mile							
	Gasoline-powered vehicles				Diesel-powered vehicles			
	Rural travel		Urban travel		Rural travel		Urban travel	
	Actual rate	Computed rate [1]	Actual rate	Computed rate [1]	Actual rate	Computed rate [1]	Actual rate	Computed rate [1]
17,000	0.150	0.152	0.175	0.189	----	----	----	----
21,300	.166	.165	.218	.207	----	----	----	----
27,000	.184	.182	.232	.230	----	----	----	----
34,500	.206	.204	.263	.261	0.176	0.167	0.147	0.146
42,000	.229	.227	.291	.292	.176	.174	.179	.169
51,200	.243	.254	.332	.330	.164	.184	.180	.198
59,500	.280	.279	.365	.364	.189	.192	.225	.224
67,900	.308	.304	.395	.399	.212	.200	.255	.250

[1] Computed rates are based on the following formulas: gasoline, rural, 0.10115+0.00299W; gasoline, urban, 0.11865+0.00413W; diesel, rural, 0.13180+0.00101W; diesel, urban, 0.03924+0.00310W. (W=GVW in thousands of pounds.)

Table 7.—Comparison of motor-fuel consumption rates of three studies of trucks operating over rural highways,[1] classified by gross vehicle weights

Average gross vehicle weight (pounds)	Motor-fuel consumption in gallons per mile from indicated studies							
	1958 five-State study					1937 Oregon study		1948 Pennsylvania study
	Gasoline-powered vehicles				Diesel-powered vehicles	Gasoline-powered vehicles	Diesel-powered vehicles	Gasoline-powered vehicles
	Engine cubic inch displacement of—			Average				
	302-406	450-549	590					
20,000	0.160	0.181	----	0.161	----	----	----	0.135
30,000	.181	.207	0.246	.191	0.156	0.203	0.128	.170
40,000	.202	.234	.262	.221	.169	.251	.157	.200
50,000	.224	.260	.279	.251	.181	.295	.183	.228
60,000	.245	.286	.295	.281	.193	----	----	.255
70,000	----	.312	.311	.311	.205	----	---	.282

[1] Rate of rise and fall for Oregon data was 1.0; for the other study data it was 1.2.

wholesale motor-fuel delivery vehicles are shown separately in figure 4 as they were not considered for this study as multi-stop city delivery vehicles.

Comparison with previous studies

Fuel-consumption rates obtained in this study have been compared with results found in two previous studies—a 1937 Oregon study (*1*), and a 1948 Pennsylvania study (*2*).[2] The comparison of these consumption rates are given in table 7 and illustrated graphically in figure 5. For comparative purposes, the average consumption rates found in the 1958 study, rather than the rates found for the individual groupings of vehicles, were used. Considering the entire gross vehicle weight range, the consumption rates obtained in the 1958 study were found to be approximately 10 percent higher than corresponding data reported in the Pennsylvania study, which were obtained by controlled tests on new vehicles. The higher motor-fuel consumption rates in commercial operation as compared with the controlled test operation can be ascribed partly to a greater prevalence of speed changes in commercial operation than had been encountered in the test truck operation, and partly to the fact that the commercial truck engines were not kept to the high degree of performance efficiency as the controlled test trucks, which were regularly maintained by factory mechanics.

It appears that the results in the Pennsylvania study, which covered a much wider range of gross vehicle weights and rates of rise and fall, may be increased by 10 percent and used to represent the fuel characteristics of vehicles now in actual commercial service.

Gasoline consumption rates in the 1937 Oregon study and the 1958 study were quite similar in the lower gross vehicle weights but the Oregon study gasoline consumption rates were higher by nearly 20 percent for the gross vehicle weights at 50,000 pounds. Diesel-fuel consumption figures in the Oregon study were lower than the 1958 study diesel consumption rates by as much as 30 percent in the lower weight ranges but were almost identical for gross vehicle weights at 50,000 pounds.

Average Time Consumption Rates

The travel time consumption rate of commercial motortrucks in rural line-haul operation was analyzed in two different ways. The first analysis was made to determine the travel time of vehicles for all trips, without considering rise and fall or traffic friction. This analysis was made in a manner similar to that used for determining the fuel-consumption rates. Actual and computed travel time consumption rates are given in table 8. In figure 6, straight lines are used to relate travel time and gross vehicle weight for each of the engine characteristic groups. It is seen that vehicles with engine displacement size of 302-406 cubic inches, which traveled

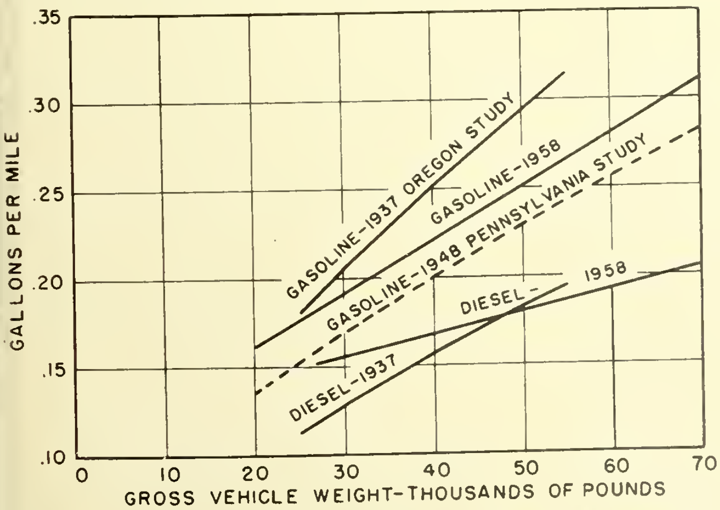

Figure 5.—Comparison of data from 3 reports on motor-fuel consumption rates of line-haul trucks.

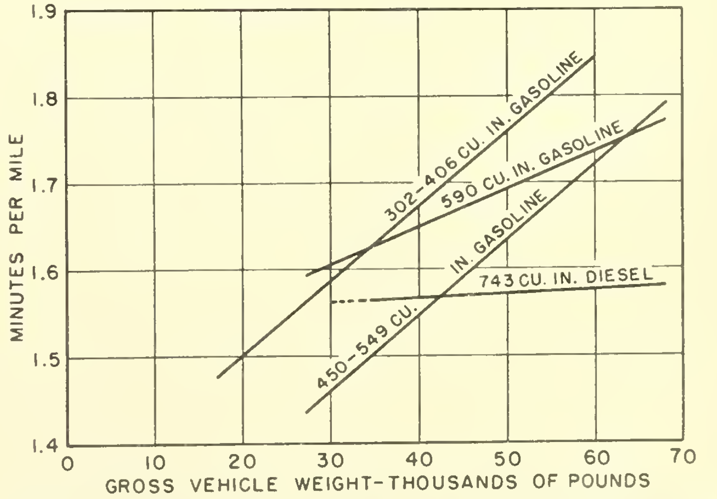

Figure 6.—Comparison of average time consumption rates of rural line-haul trucks by engine size for 1.2 rise and fall.

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[2] Italic numbers in parentheses refer to the list of references on p. 31.

Table 8.—Rates of travel time consumption for trucks in rural line-haul service in 5 States, 1957–58 [1]

Gross vehicle weight (pounds)	Travel time-consumption rates in minutes per mile for vehicles with engine displacement of—								
	302–406 cu. in., gasoline (140 h.p.) engine		450–549 cu. in., gasoline (171 h.p.) engine		590 cu. in., gasoline (225 h.p.) engine		Gasoline engine average—Actual rate	743 cu. in., diesel (200 h.p.) engine	
	Actual rate	Computed rate [2]	Actual rate	Computed rate [2]	Actual rate	Computed rate [2]		Actual rate	Computed rate [2]
17,000	1.434	1.478	------	--------	------	--------	1.434	------	--------
21,300	1.506	1.514	------	--------	------	--------	1.506	------	--------
27,000	1.649	1.563	1.467	1.436	1.606	1.593	1.592	------	--------
34,500	1.619	1.627	1.520	1.501	1.590	1.626	1.596	1.636	1.567
42,000	1.687	1.691	1.526	1.566	1.662	1.659	1.620	1.460	1.571
51,200	1.728	1.769	1.626	1.645	1.738	1.699	1.692	1.616	1.576
59,500	1.859	1.840	1.660	1.717	1.724	1.735	1.696	1.569	1.580
67,900	----	--------	1.859	1.790	1.761	1.771	1.797	1.598	1.585
Average	1.638	--------	1.586	--------	1.696	--------	1.625	1.559	--------

[1] Average rate of rise and fall, 1.2 feet per 100 feet.

[2] Computed rates are based on the following formulas: 302–406 cu. in., 1.333+0.008516W; 450–549 cu. in., 1.203+0.008642W; 590 cu. in., 1.476+0.004347W; and 743 cu. in., 1.549+0.000526W.

Table 9.—Summary of travel time consumption rates, by gross vehicle weights and power characteristics, for *urban* line-haul freight vehicles and city delivery vehicles

Time-consumption rates in minutes per mile for urban line-haul vehicles with engine displacement of—						Time cousumption rates in minutes per mile for city delivery vehicles	
Gross vehicle weight (pounds)	302–406 cu. in., gasoline engine	450–549 cu. in., gasoline engine	590 cu. in., gasoline engine	Average gasoline engine	743 cu. in., diesel engine	Gross vehicle weight (pounds)	All gasoline engines
17,000	3.207	2.868	------	3.105	------	4,600	8.854
21,300	2.909	3.473	------	2.987	------	6,000	5.736
17,000	3.388	2.818	2.957	3.182	2.556	10,500	6.181
34,500	3.260	2.973	2.378	3.136	2.353	14,500	5.125
42,000	3.274	3.082	2.728	3.167	2.597	18,500	4.500
51,200	3.513	2.914	2.283	3.253	2.901	22,500	5.502
59,500	4.533	2.987	2.532	3.435	3.043	27,500	4.847
67,900	4.486	2.630	2.815	3.039	2.784	33,300	4.184
Average	3.306	2.997	2.671	3.156	2.740	Average	5.443

at a rate of 1.59 minutes per mile with a GVW of 30,000 pounds, traveled at 1.85 minutes per mile when the GVW was 60,000 pounds. Vehicles in the 450–549-cubic-inch engine size group traveled 1.46 and 1.72 minutes per mile at corresponding weights. The straight-line relationships for these two engine groups were approximately parallel, indicating a constant rate of increase in travel time consumed with increase in gross vehicle weights.

The vehicles with 743-cubic-inch diesel engines maintained a much more constant speed with respect to gross vehicle weights than those with the larger gasoline engines, showing an increase of only 0.02 minute per mile from 30,000 to 60,000 pound GVW.

The travel time consumption rates of commercial vehicles in urban line-haul and in city pickup and delivery service are shown in table 9. Although time-consumption rates were not found to vary in a uniform manner with gross vehicle weight, it was noted that as the power characteristics of engines increased the time consumption decreased.

Table 10.—Average speeds of gasoline- and diesel-powered trucks, experiencing less than two slowdowns per mile and no stops in Ohio and Washington rural line-haul operation [1]

Gross vehicle weight (pounds)	Time-consumption rates [2]							
	302–406 cu. in. gasoline engine		450–549 cu. in. gasoline engine		590 cu. in. gasoline engine		743 cu. in. diesel engine	
	Minutes per mile	Miles per hour	Minutes per mile	Miles per hour	Minutes per mile	Miles per hour	Minutes per mile	Miles per hour
17,000	1.34	44.8	------	------	------	------	------	------
21,300	1.38	43.5	------	------	------	------	------	------
27,000	1.43	42.0	1.35	44.4	1.36	44.1	------	------
34,500	1.50	40.0	1.40	42.9	1.38	43.5	1.483	40.5
42,000	1.57	38.2	1.44	41.7	1.40	42.9	1.486	40.4
51,200	1.65	36.4	1.50	40.0	1.43	42.0	1.490	40.3
59,500	----	----	1.56	38.5	1.45	41.4	1.493	40.2
67,900	----	----	1.61	37.3	1.47	40.8	1.497	40.1

[1] Average rate of rise and fall, 1.3 feet per 100 feet.

[2] Rates were computed by the following formulas. 302–406 cu. in.: m.p.m., 1.18935+0.00916W; m.p.h., 49.1986−0.25747W.
450–549 cu. in.: m.p.m., 1.17435+0.00643W; m.p.h., 49.2757−0.17956W.
590 cu. in.: m.p.m., 1.2909+0.00264W; m.p.h., 46.0567−0.07742W.
743 cu. in.: m.p.m., 1.4696+0.00040W; m.p.h., 41.1719−0.01905W. (W = GVW in thousands of pounds.)

Referring to the average time-consumption rates for all gasoline-powered vehicles (tables 8 and 9) it will be seen that vehicles in rural line-haul service traveled at an average rate of 1.625 minutes per mile, or 36.9 miles per hour; vehicles in urban line haul traveled at 3.156 minutes per mile or 19.0 miles per hour; and all city pickup and delivery vehicles at 5.443 minutes per mile or 11.0 miles per hour. Similar figures for diesel-powered vehicles were 1.559 minutes per mile, or 38.5 miles per hour for rural line-haul operation, and 2.740 minutes per mile, or 21.9 miles per hour for urban line-haul operation.

Average speeds in free-flowing traffic

The second analysis made of travel time for rural line-haul operations involved the desired speeds at which vehicles traveled in free-flowing traffic when they apparently were unrestricted except by speed limits or safe driving speeds. It was possible to study the speeds by analyzing time-consumption rates on certain highway sections in Ohio and Washington where trucks traveled without experiencing more than two slowdowns per mile and no stops. The average operating speeds under these conditions were related to the four groupings of engine sizes and power characteristics and to gross vehicle weight, as shown in table 10 and figure 7.

Travel time, in minutes per mile, increased sharply as the gross weight of gasoline-powered commercial trucks in the lowest range of engine size and power increased. Conversely of course, average road speeds decreased sharply. However, as the engine horsepower and gross vehicle weight increased, the travel time increase was less pronounced. This is reflected by the steepness of the slope of the lines per 10,000-pound increase in GVW. For the lowest gasoline-powered engine size, the rate increased 0.09 minute per mile for each increase of 10,000 pounds in GVW. For the medium gasoline-powered engine size, the corresponding increase was 0.06 minute per mile and for the 590-cubic-inch engine gasoline-powered vehicles and the diesel-powered vehicles the increases were 0.03 and 0.01 minute per mile, respectively.

The relative performance of the four groupings of vehicles (fig. 7) point up the consideration that while better fuel economy is attained with smaller engines for the gross vehicle weights investigated, the penalty of using smaller engines is an increase in travel time consumption at higher vehicle weights.

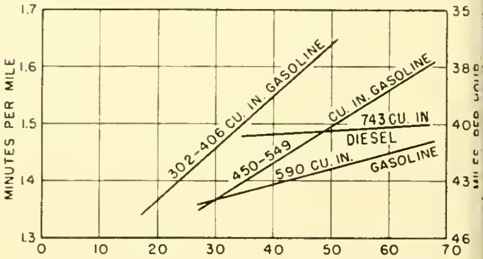

Figure 7.—Average time consumption rates for trucks operating in free-flowing traffic on rural line-haul service with an average rate of rise and fall of 1.3 feet per 100 feet.

Time-consumption rates compared

Another important use of the current study data was in comparison with the average time-consumption rates reported in the 1948 Penn-ylvania study (*2*). Travel-time-consumption ates are graphically illustrated for the two tudies in figure 8, using the average rates for all vehicles.

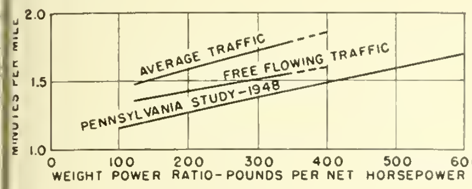

Figure 8.—Comparison of 1957–58 travel time rates for rural line-haul gasoline-powered trucks for 1.2 rate of rise and fall with 1948 Pennsylvania study data based on 1.3 rate of rise and fall.

The time-consumption rates obtained in the 1958 study, considering the average travel time for all conditions of traffic, are labeled "average traffic" in figure 8 and were found to be 26 percent higher than corresponding data reported in the Pennsylvania study. A comparison of greater significance, however, can be made between the 1958 study, identified in igure 8 as "free-flowing traffic," and those of 1948 Pennsylvania study, because both were nade under similar conditions. The time-consumption rates of gasoline-powered trucks raveling in free-flowing traffic were 10 percent higher than corresponding data reported n the Pennsylvania study.

Effect of Traffic on Performance

One of the main objectives of the study was to investigate the effect of varying traffic volumes on the performance of commercial vehicles. Other studies (*1—4*) have made a good start in determining the fuel consumption and travel time for uniform speeds, stops and starts, and slowdowns; and in finding out how certain factors, such as gradient, rise and fall, horizontal curvature, gross vehicle weight, and engine characteristics, effect fuel and time consumption. However, little has been available in the literature as to the effect of varying traffic volumes.

It was hoped that this study would provide a means for estimating the added operating cost brought about by frictions in the traffic stream. The basic approach was one of considering the number of speed changes per mile for varying volume conditions, the percentage of the total number of speed changes that were stops and starts, and the average speed change in terms of miles per hour of a stop or slowdown. It was reasoned that if such information could be provided, the added cost for having to operate other than at a uniform speed could readily be assessed.

Speed changes per mile

What are probably the most significant results of this study, speed changes per mile, were computed for trucks with different gross weights operating over three types of rural highways with varying average daily traffic and are shown in table 11. An attempt was made to develop similar data for urban operation, but the lack of traffic data for the irregular routes traveled made this impossible.

The average values of speed changes per mile (from table 11) are shown in figure 9 as straight-line relationship established for the three types of highways. The benefits accruing from the elimination of impediments to free-flowing traffic are clearly illustrated by comparing the speed changes per mile on the 4-lane divided, controlled-access facility with those on the 4-lane undivided, uncontrolled-access facility. For an average daily traffic of 15,000 vehicles, there were an average of 2.0 speed changes per mile on the 4-lane uncontrolled-access highway as compared with a rate of about 0.8 on the 4-lane controlled access highway. Speed changes per mile on 2-lane highways increased from 2.0 to 2.8 where the average daily traffic increased from 5,000 to 10,000. In contrast, speed changes per mile on the 4-lane uncontrolled-access highway increased from 1.5 to 1.8 over the same average daily traffic range.

Data for 4-lane divided highways with no access control were not obtained in sufficient quantity for analysis. It is reasonable to expect that the relationship for this type of highway would fall between that for the two 4-lane highways shown in figure 9, and would probably lie closer to the 4-lane undivided, uncontrolled-access highway.

Analysis of speed changes

Of considerable importance among the data obtained were the percentages of total speed changes representing stops and slowdowns. Speed changes caused by stops and slowdowns are presented in table 12 from results of the studies made in Ohio and Washington, the only States where stops were recorded. On the average, complete stops occasioned about 11 percent of the speed changes in rural line-haul operations and about 45 percent in urban line-haul operations.

Compiled from the limited data available, an analysis of speed changes in miles per hour was made and it was found that an average stop in rural areas was made from a speed of 26 miles per hour. On city streets the average stop was made from a speed of 18.9 miles per hour. The average change in speed for slowdowns in both rural and urban areas was 11.4 miles per hour.

To illustrate the significance of a speed change in terms of motor-fuel consumption and to confirm that fuel consumption increases with an increasing number of speed changes per mile, gasoline-consumption rates were computed for road sections having different rates of speed change per mile, for different gross vehicle weights. The average rates are presented in table 13 for the three types of operation.

The straight-line relationships established for the data in table 13 are shown in figure 10. It is seen that an increase of one speed change

Table 11.—Speed changes per mile made by trucks operating over three types of rural highways with varying average daily traffic

Average daily traffic on indicated highways	Highway section mileage	Number of trips	Total miles traveled	Speed changes per mile for vehicles with average gross vehicle weight (1,000 pounds) of —						
				17.0	23.1	27.4	36.3	43.7	52.2	Average
4-lane divided, controlled access:										
46,700	6.56	54	354.24	1.56	2.05	------	2.38	2.71	2.36	2.19
23,300	3.62	54	195.48	.62	.83	------	1.17	1.90	1.99	1.24
12,700	8.19	54	442.46	.38	.62	------	.53	.66	.74	.60
4-lane undivided, uncontrolled access:										
15,700	31.71	54	1,712.34	1.73	1.79	------	2.12	2.35	2.34	2.03
10,300	48.78	54	2,634.12	1.59	1.55	------	2.07	2.03	2.12	1.82
5,200	5.67	52	153.79	------	1.19	------	1.58	1.47	1.73	1.53
2-lane:										
8,800	27.30	56	1,528.80	------	------	2.75	2.74	2.61	2.69	2.71
6,000	57.58	56	3,224.48	------	------	2.24	2.14	2.15	2.31	2.18
2,000	20.52	56	1,149.12	------	------	1.59	1.48	1.35	1.70	1.50

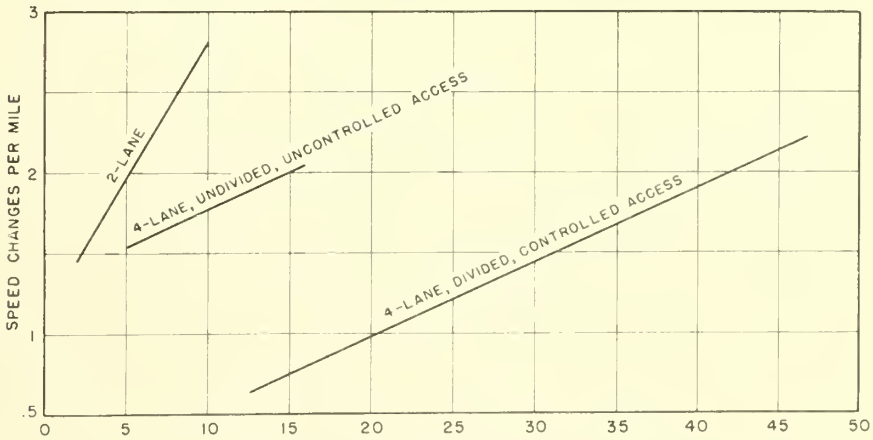

Figure 9.—Average speed changes per mile for rural line-haul trucks, by average daily traffic and type of highway.

Table 12.—Number and percentage of speed changes occasioned by slowdowns and stops of trucks in Washington and Ohio rural and urban line-haul travel

Speed Changes	Washington			Ohio			Total		
	Slow-downs	Stops	Speed changes	Slow-downs	Stops	Speed changes	Slow-downs	Stops	Speed changes
Rural line-haul:									
Number	5, 358	795	6, 153	8, 036	935	8, 971	13, 393	1, 731	15, 124
Percent	87. 1	12. 9	100. 0	89. 6	10. 4	100. 0	88. 6	11. 4	100. 0
Urban line-haul:									
Number	1, 220	613	1, 833	1, 581	1, 688	3, 269	2, 801	2, 301	5, 102
Percent	66. 6	33. 4	100. 0	48. 4	51. 6	100. 0	54. 9	45. 1	100. 0

per mile for a vehicle weighing 30,000 pounds traveling on a rural highway resulted in an average fuel-consumption increase of 0.010 gallon per mile. The corresponding increase for vehicles in urban line-haul operation was 0.021 gallon per mile, and for city delivery vehicles the average increase was 0.0056 gallon per mile. The greater rate of speed change for urban line-haul operation as compared to rural line-haul operation is probably due to the higher incidence of stops and slowdowns. City pickup and delivery vehicles consume less gasoline per speed change than the urban line-haul vehicles because stops and slowdowns are of lesser magnitude, as evidenced by an average speed of 11 miles per hour.

Also of importance is the indication that fuel consumption attributable to a speed change increases with gross vehicle weight. For example, the fuel consumed for an increase of one speed change per mile for rural line-haul operations was 0.0092 gallon for vehicles with 20,000 pounds GVW and 0.0142 gallon for 50,000 pounds.

Data for travel time-consumption rates due to one speed change per mile were also developed, and are shown in table 14. The average time-consumption rate did not appear to increase with gross weight but the average value for all gross vehicle weights increased as the speed changes per mile increased.

The average time consumed in one speed change for rural line-haul operation was found to be 0.26 minute, or 15.6 seconds; for urban line-haul operation 0.27 minute, or 16.2 seconds; and for city pickup and delivery operation 0.38 minute, or nearly 23 seconds. In spite of the fact that the speeds from which stops and slowdowns were made were higher in rural than in urban line-haul operation, the time consumption per speed change is about equal, probably because the percentage of total speed changes that are stops is much higher in the urban line-haul.

The increased fuel- and time-consumption rates for one speed change have been developed principally for illustrative purposes, although they can be used in estimating benefits. When data are available from controlled tests (*3*—*4*) on a variety of vehicles, the data herein presented may be refined.

Table 13.—Gasoline-consumption rates for trucks in line-haul and city pickup and delivery operation for various rates of speed change per mile

Average gross vehicle weight (*pounds*)	Gasoline-consumption rates in gallons per mile for indicated number of speed changes per mile						
	1	3	4	5	7	9	12
Line-haul, rural:							
17,000	0. 134	0. 142	------	0. 160	0. 181	------	------
34,500	. 180	. 198	------	. 226	. 250	------	------
42,000	. 200	. 222	------	. 255	. 279	------	------
53, 000	. 228	. 257	------	. 300	. 322	------	------
57,000	. 239	. 270	------	. 311	------	------	------
68,000	. 268	. 305	------	------	------	------	------
Average	. 197	. 220	------	. 251	. 279	------	------
Line-haul, urban:							
17,000	0. 143	0. 149	------	0. 153	------	------	------
26,000	. 159	. 180	------	. 198	------	0. 324	------
28,000	------	------	------	------	0. 246	------	------
52,000	. 206	. 268	------	. 328	. 409	. 426	------
58,000	. 217	------	------	------	------	------	------
59,000	------	. 292	------	------	. 457	------	------
61,000	------	------	------	. 373	------	------	------
62,000	------	------	------	------	------	. 465	------
Average	. 185	. 224	------	. 269	. 333	. 382	------
City pickup and delivery:							
6,000	------	------	0. 111	------	------	------	0. 145
10,500	------	------	. 131	------	------	------	. 167
18,500	------	------	. 165	------	------	------	. 206
27,500	------	------	. 204	------	------	------	. 250
33,300	------	------	. 229	------	------	------	. 279
Average	------	------	. 143	------	------	------	. 168

Cost of a Speed Change

The approximate cost of a stop is included in this article more as a matter of interest than with the idea of establishing valid cost values. Many sections of rural highway studied were traveled by line-haul vehicles without experiencing any stops and with less than two slowdowns per mile. Likewise, certain urban sections of highway studied were traveled by line-haul vehicles with a high incidence of stops but with less than two slowdowns per mile.

In order to estimate the cost of a stop the entire fuel consumption rate for the rural travel with no stops was subtracted from the fuel-consumption rate for urban travel where a high incidence of stops occurred. The difference is attributed solely to the effect of stops because slowdowns were the same in both instances. It should be remembered though, that the average stop was made from 26 miles per hour in rural areas and 19 miles per hour in urban areas. Dividing the total consumption per mile due to traffic stops by the number of stops per mile gave the consumption rates per stop which are shown in table 15. Gasoline consumed per stop showed a definite increase as the GVW increased. For example, if a cost per gallon of fuel of 30 cents is used, the cost of a stop would range

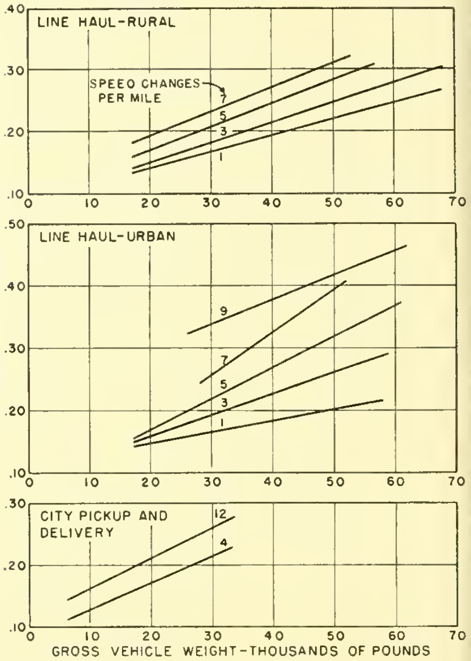

Figure 10.—Gasoline consumption rates, by rate of speed change per mile and by gross vehicle weight, for line-haul and city delivery vehicles.

ble 14.—Average travel time-consumption for trucks in line-haul and city pickup and delivery operations for various rates of speed changes per mile

Type of travel	Average time consumption in minutes per mile for the indicated number of speed changes per mile							Average time lost per speed change (minutes)
	1	3	4	5	7	9	12	
Rural	1.48	1.89	------	2.33	3.05	------	------	0.26
Urban	2.35	2.69	------	3.20	3.81	4.53	------	.27
City pickup and delivery	------	------	4.39	------	------	------	7.43	.38

om one-half cent for a GVW of 17,000 pounds
more than 2 cents for a GVW of 51,000
unds.

Knowing the number of speed changes
ved, the proportion of stops and slowdowns,
d the magnitude of each, it is possible to
mpute the added cost of fuel and travel time
a speed change if the extra fuel and time
nsumed during the speed change is known.
ius, using speed changes per mile as a
easure of congestion, the benefits may be
mputed that accrue from highway improve-
ents that reduce congestion. It is realized
at at present the tool is rough, but it can be refined. This is planned, using digital recorders instead of human observers.

References

(1) Beakey, John

The effect of highway design on vehicle speed and fuel consumption. Oregon State Highway Commission Technical Bulletin No. 5, Salem, 1937.

(2) Saal, Carl C.

Time and gasoline consumption in motor truck operation. Highway Research Board, Research Report No. 9-A, 1950.

Table 15.—Gasoline consumption rates for trucks in line-haul operation due to traffic stops, by gross vehicle weight

Gross vehicle weight (pounds)	Gallons per stop	
	Actual rate	Computed rate [1]
17,000	0.014	0.017
21,300	.030	.024
27,000	.034	.034
34,500	.044	.046
42,000	.054	.058
51,200	.076	.073

[1] Computed from straight line formula 0.001625W−0.0103. (W=GVW in thousands of pounds.)

(3) Claffey, Paul J.

Time and fuel consumption for highway-user benefit studies. See pp. 16–21 of this issue of Public Roads.

(4) Sawhill, Roy B.

Motor transport fuel consumption rates. Highway Research Board paper, 39th Annual Meeting, Jan. 1960.

New Publication: Highway Progress, 1959

The Annual Report of the Bureau of Public Roads, Fiscal Year 1959, newly titled *Highway Progress, 1959,* is now available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C. at 40 cents a copy.

Reflecting the central role of highway transportation in our entire way of life, the greatly expanded construction program launched by the Federal-Aid Highway Act of 1956 continued its vigorous growth during the fiscal year.

Highway usage also increased, with motor-vehicle registrations forecast to reach a record high of 70.4 million in the calendar year 1959, and travel by these vehicles pointing toward at least 700 billion vehicle miles.

The Bureau of Public Roads had set a fiscal year goal of $3.075 billion in Federal-aid obligations for surveys and plans, right-of-way acquisition, and construction. Actual obligations of the year reached $3.223 billion, as compared with a $2.749-billion total for the previous year.

During the fiscal year 1959, completions of all classes of Federal-aid and Federal projects provided improvements on 32,828 miles of roads and streets—with the growing use of multilane highways—that total represented the equivalent of 71,336 miles of single-lane construction. Individual projects included examples of the most spectacular and the most complex construction in highway history. Construction put in place during the year called for $2.875 billion of Federal funds, an increase of 74 percent over the previous year.

Projects for the construction of 30,923 miles of improvements also were set up during the year under the Federal-aid and Federal highway programs.

These extensive Federal-aid operations were supported largely with funds authorized by the Federal-Aid Highway Acts of 1956 and 1958. On August 1, 1958, Federal-aid funds for fiscal year 1960, amounting to $3.5 billion, were apportioned to the States. This brought the total of Federal-aid apportioned since passage of the 1956 Act to $10.55 billion.

Federal-aid improvements under the continuing primary, secondary, and urban programs progressed on a larger scale than ever before, but the Interstate System held its position as the focal point of public interest in highways. As sections were opened to traffic, more and more motorists began experiencing the advantages of the controlled-access freeway with its relief from cross traffic, its greater comfort and safety, and significant savings in travel time and vehicle operating costs. Residential, commercial, and industrial centers developed adjacent to the Interstate right-of-way—further indication of the catalytic role of motor transport.

Locating such important highway facilities was not left to chance. These complex Interstate System projects usually require several years of preparatory work—planning and surveying, plus design and right-of-way acquisition—before actual construction begins. The progress report on the Federal-aid highway program, presented to Congress during the fiscal year, showed that by December 31, 1958, improvements substantially meeting approved standards and at least adequate for current traffic, had been completed on 4,831 miles of the Interstate System and that during the calendar year 1958, there were 2,078 miles of high type pavement placed under construction.

At the end of the fiscal year progress of the Interstate System program was threatened by two financial problems. The first was immediate, and eminated from a sharply accelerated rate of spending called for by th Federal-Aid Highay Act of 1958.

Under the basic 1956 Act, the Federal-ai program is financed on a pay-as-you-go bas from Federal highway-user taxes which g into the highway trust fund. Net income the trust fund during the fiscal year wa $2.2 billion while expenditures from the fun for Federal-aid highways amounted to $2. billion. This was the first year that expen itures had exceeded receipts. Apportio ments for fiscal 1960, made under the 195 Act, considerably outweighed anticipate revenues to the trust fund. It was recognize that this action would deplete the surpl accumulated through earlier years of th trust fund and would preclude apportionme of any funds for the Interstate System for th fiscal year 1961. As the fiscal year 195 ended, congressional action to remedy th situation was pending.

There was, in addition, a long-range prob lem of financing the Interstate System pr gram. The new estimate of the cost of com pleting the Interstate System presented t the Congress during fiscal year 1958 showe that Federal and State matching financin required after July 1, 1956, amounted t $37.6 billion, as compared with the $27. billion figure contemplated in the Feder Aid Highway Acts of 1954 and 1956.

During fiscal year 1959 the Bureau of Publi Roads submitted the *Third Progress Report* *the Highway Cost Allocation Study.* Schedule for presentation to Congress early in 196 together with a new detailed estimate of th cost of completing the Interstate System, it expected that these reports will be of re assistance to the Congress in its consideratio of appropriate scheduling and financing Interstate and other Federal-aid highwa programs.

Errata

The table *State Legal Maximum Limits of Motor Vehicle Sizes and Weights Compared With AASHO Standards,* which appears on pages 250–251 of Public Roads magazine, vol. 30, No. 11, December 1959, requires a correction in footnote references.

In the entries for Maryland and Ohio, the footnote references 31 and 44 should be deleted from the column "Axle load-pounds, Tandem, Statutory limit." In the entry for North Dakota, footnote reference 44 should be added in the "5-axle" and "Other combination" columns under the heading "Practical maximum gross weight."

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JUNE 1960

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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IN THIS ISSUE: Economic costs of motor-vehicle traffic accidents.

Public Road

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U.S. DEPARTMENT OF COMMERCE
FREDERICK H. MUELLER, Secretary

BUREAU OF PUBLIC ROADS
BERTRAM D. TALLAMY, Administrator
ELLIS L. ARMSTRONG, Commissioner

A Symposium on Traffic Accident Costs

In this issue, major findings of motor-vehicle accident cost studies conducted in Massachusetts and Utah are discussed. Cost data are classified on the basis of severity of accidents and are further related to the activities of persons involved in accidents, to the highway systems on which the accidents occurred, and to the types of vehicles involved.

Placing a dollar value on losses resulting from traffic accidents in no way minimizes the personal tragedy of either traffic fatalities or serious injuries; it is simply a means of identifying and measuring financial losses that in turn can be used as a tool in the planning of both highway safety and highway improvement programs.

Of the four articles presented here, three report findings of the Massachusetts accident cost study and the fourth applies to a similar study made in Utah. The findings are based on data from only two States, and whether similar patterns or trends obtain n other States can only be determined when studies now under way or planned for he future are completed. The data presented, however, are believed to be generally indicative of what may be expected in other States of similar characteristics.

In this analysis, economic costs are confined specifically to direct costs of traffic accidents. Other direct costs in connection with non-traffic accidents and traffic and non-traffic incidents are excluded. Non-traffic accidents are those involving motion but not occurring on a public roadway. Traffic and non-traffic incidents are mishaps occurring on and off public roadways, not involving motion, and attributable to vandalism, fire, and acts of God.

Direct costs are composed of the money value of damage to property; hospitilization; services of physicians, dentists, and nurses; ambulance use; medicine; work ime lost; damages awarded in excess of other direct costs; attorneys' services; court fees; and other lesser items.

Another large segment of costs borne by he public are indirect costs. Although not a part of this analysis, they must be considered in any total view of accidents. They include such items as loss of future earnings as a result of injuries causing death or permanent disability, overhead cost of motor-vehicle accident insurance as well as certain other types of insurance with accident coverage, property damage indirectly resulting from an accident, illness or death of a person not involved in an accident but traceable thereto, public aid for care of indigent persons injured in motor-vehicle accidents, and expenditures for driver training and other accident prevention activities.

The economic costs of motor-vehicle accidents are discussed from four aspects: First, the cost in relation to persons involved in accidents; second, the cost in relation to the road systems; third, the cost in relation to passenger cars and trucks; and fourth, the cost in relation to highway planning and improvement.

The Economic Cost of Traffic Accidents in Relation to the Human Element *(p. 34) discusses the impact of traffic accidents on the total population of Massachusetts. It shows that if existing traffic accident rates are not reduced, every individual in the entire population of Massachusetts would be in a motor-vehicle accident at least once every 13 years, or 5 times during his life. The article points out, however, that the possibilities of being killed or seriously injured are very slight.*

Of the 4.8 million residents of Massachusetts in 1953, less than 8 percent were involved in traffic accidents.

Of the 45,472 persons injured, 421 were fatally injured, 18,045 were seriously injured, and 27,006 were superficially injured. Injuries sustained in accidents caused 19,405 persons to lose an average of 19 days worktime per person; they hospitalized 6,647 persons for an average of 9.4 days each. Traffic accident injuries permanently reduced to some degree the ability of 2,117 persons to earn a living, and they similarly temporarily affected another 3,514 persons.

The total direct cost of traffic accidents in Massachusetts in 1953 was more than $50 million, of which fatal injury accidents cost $1.6 million; nonfatal injury accidents, $28.7 million; and property-damage-only accidents, $19.9 million.

This article points up one significant thing—of the total economic loss from traffic accidents, 59 percent was due to property damage alone, and 41 percent to injuries and related costs. The total direct cost of accidents was divided among six major elements: Property damage, $27.6 million; treatment of injuries, $3.4 million; worktime lost, $4.5 million; loss of use of vehicle, $0.4 million; legal and court fees, $5.2 million; and awards and settlements, $9.1 million.

The Economic Cost of Accidents in Relation to the Highway Systems *(p. 39) evaluates the efficiency of the Massachusetts highway systems from the standpoint of economic losses through automobile accidents. A major finding was that of the 131,536 traffic accidents in Massachusetts during 1953, 87 percent occurred in urban areas. Likewise, 87 percent of the direct costs were from accidents occurring in urban areas.*

Classified by highway systems, the cost of fatal accidents was $662,810 on the Federal-aid primary system, $227,480 on the Federal-aid secondary system, and $751,460 on local roads not a part of the Federal-aid systems.

In the Federal-aid primary system, State highways had a decidedly lower average accident rate than local roads. Conversely, in the Federal-aid secondary system the State highways had a much higher accident rate than local roads.

In addition to these general system findings, the article demonstrates that highways with full control of access experienced much lower accident rates, at a given traffic volume, than did those highways where access was not controlled.

The Economic Cost of Traffic Accidents in Relation to the Vehicle *(page 44) compares the accident experience of passenger cars and trucks registered in Massachusetts. A further comparison is made of trucks grouped by visual classification and by gross weight classification.*

A significant finding was that the cost of accidents per mile of travel for passenger cars was nearly double that of trucks, with the single exception of cost per mile for fatal accidents. It was also found that both the accident involvement rate and the accident cost per mile for passenger cars increased with the age of the car, but this was not true of trucks.

Truck-tractor-semitrailer combinations had the best accident record among the truck groups, on the basis of exposure, with the sole exception of a higher fatal involvement rate and fatal accident cost per mile. Panel and pickup trucks, together with the 2-axle, 6-tire trucks, accounted for 82 percent of the truck registrations, 73 percent of the truck travel, 74 percent of all truck involvements in accidents, and 80 percent of the truck accident costs.

An important relation developed with regard to registered gross weight was the decrease of accident cost per mile as the gross weight increased. Both the involvement rate and accident cost per mile of the heaviest group of trucks were the most favorable when compared to those of all other trucks.

The Economic Cost of Traffic Accidents in Relation to Highway Planning *(p. 49) points up the value of traffic accident cost data to the highway planner in evaluating and scheduling highway improvement.*

The article also compares the frequencies and costs of traffic accidents in Utah in 1955 with the results found in the 1953 Massachusetts study.[1] *Despite dissimilarities in geographic characteristics and population densities, the relative distributions of accidents in the two States were quite consistent. As to costs, regardless of severity or type, accidents were more costly in Utah than in Massachusetts.*

---

[1] *The Economic Costs of Motor-Vehicle Accidents of Different Types*, by Robie Dunman, PUBLIC ROADS, vol. 30, No. 2, June 1958. The basic data from that article are included in the tables on p. 50 of this issue.

The Economic Cost of Traffic Accidents in Relation to the Human Element

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by [1] ROBIE DUNMAN
Transportation Economist

EXTENSIVE use of traffic accident cost data developed by the Massachusetts Department of Public Works and by the Massachusetts Registry of Motor Vehicles in cooperation with the U.S. Bureau of Public Roads was made in the Congressional report *The Federal Role in Highway Safety*.[2] These data have also been used as the basis of other reports relating economic costs to accident types, characteristics of the street and highway systems, and to the characteristics of motor vehicles. This article, using data from the same source, relates the economic cost of motor-vehicle traffic accidents to persons.

Comparisons involving passenger cars of Massachusetts registry in 1953 are made of the accident experience of automobile drivers, passengers, pedestrians, and other persons involved in motor-vehicle traffic accidents. The number of accidents, the number of persons involved, the number of persons injured fatally, seriously, and superficially, the number hospitalized, and the number permanently and temporarily disabled are discussed. Equally important, the cost of accidents and injuries is revealed.

Injury rates and injury cost rates, number of persons injured per 100,000 population, and the per capita cost of their injuries show the relative economic importance of the accidents and injuries experienced by each class of persons.

The Massachusetts study of the economic cost of motor-vehicle accidents encompassed the total driving experience of all licensed operators of passenger cars and cargo-carrying vehicles of Massachusetts registry, including those who experienced accidents and those who did not, in the operation of passenger cars during 1953, within or outside the State, and in the operation of cargo-carrying vehicles during 1955. However, the discussion in this article is confined to the accident experience of Massachusetts licensed passenger-car operators and persons in motor-vehicle traffic accidents involving passenger cars on public streets and highways of the State. This article excludes accident experience of truck operators, the number and cost of accidents occurring on private property, and the number and cost of mishaps involving acts of vandalism and acts of God.

Statistical studies of the motor-vehicle accident costs were based on a probability sample designed to be accurate within 10 percent. To determine the number of persons involved in fatal accidents, original accident reports were reviewed and the number of persons reported represents a firm figure. The number of drivers involved in nonfatal injury and property-damage-only accidents was unavoidably slightly inflated because a driver who was involved in more than one accident during the year was counted as one driver each time he was involved in an accident. The number of passengers involved in accidents was estimated on the basis of an average passenger-car occupancy of 1.6 persons. The number of pedestrians and "other" persons nonfatally injured, however, was obtained from the original accident reports.

Costs reflected in this article are direct costs and consist exclusively of the money value of damage to property, injuries to persons, worktime lost, loss of use of vehicle, legal and court costs, damage awards in excess of actual cost, and small miscellaneous items. The number of persons losing worktime includes only those employed in gainful occupations. The number of workdays lost includes time lost for treatment of injuries, for convalescence, and for settlement of claims.

The activity class, "others," used throughout this article, consists of persons using conveyances other than motor-vehicles—such as bicycles, horsedrawn vehicles, and trolley cars.

Physical injury that impaired earning capacity is referred to as disability. The degree of disability as used is a measure of the extent to which persons injured in motor-vehicl traffic accidents were handicapped in making a living. There are two classes of disability—temporary disability and permanent disability Under the temporary disability class, there are two degrees of disability—total and partial The degree of permanent disability is expressed in percentages of 25, 50, 75, and 100

The data regarding persons permanently and temporarily disabled as a result of motor-vehicle traffic accidents in Massachusetts appeared to be incomplete and therefore no usable in this analysis. However, rathe than include no information on this importan part of the problem, an approximation wa derived by applying to the number of person injured in Massachusetts, the percentage o persons injured that were disabled in th State of Utah, where a similar study wa being conducted.

Persons in Accidents

In 1953, the population of Massachusett was 4,773,000. Of this total, 362,280 person were involved in 131,536 motor-vehicle traffi accidents. Table 1 shows the number o accidents and their severity, the number o persons in accidents, the severity of thei injuries, and their activity at the time of th accident.

The most significant point drawn from tabl 1 was the great number of persons involve and the relatively small number of person killed and injured in motor-vehicle traff accidents. The 362,280 persons involved i

Table 1.—Number of persons in traffic accidents involving passenger cars, classified b severity of accident and activity

Item of comparison	Severity of accident							
	Fatal		Nonfatal injury		Property-damage-only		All accidents	
	Number	Percent of total	Number	Percent of total	Number	Percent of total	Number	Percent of total
Involvements:								
Severity of injuries								
Persons fatally injured	421	58.6	----------	------	----------	------	421	0.1
Persons nonfatally injured	168	23.4	44 883	47.9	----------	------	45,051	12.4
Persons not injured	129	18.0	48,751	52.1	267,928	100.0	316,808	87.5
Total persons involved	718	100.0	93,634	100.0	267,928	100.0	362,280	100.0
Activity of persons:								
Drivers	344	47.9	54,260	57.9	167,455	62.5	222,059	61.3
Passengers	206	28.7	32,556	34.8	100,473	37.5	133,235	36.8
Pedestrians	158	22.0	5,735	6.1	---	----	5,893	1.6
Others	10	1.4	1,083	1.2	----------	------	1,093	.3
Total persons involved	718	100.0	93,634	100.0	267,928	100.0	362,280	100.0
Accidents:								
Number of accidents	315	------	33,270	------	97,951	------	131,536	------
Persons per accident	2.3	------	2.8	------	2.7	------	2.7	------
Persons injured per accident	1.9	------	1.3	------	----------	----	.3	------

---

[1] Presented at the 39th Annual Meeting of the Highway Research Board, Washington, D.C., January 1960.

[2] House Document No. 93, 86th Congress, 1st Session, 1959.

motor-vehicle traffic accidents during 1953 represented one out of every 13 persons, or 7.6 percent of the total population. However, of all the persons involved in motor-vehicle traffic accidents only 45,472, or 12.5 percent, were either fatally or nonfatally injured. The vast majority of persons involved, 316,808 (87.5 percent), experienced no injury whatever.

In the distribution of persons involved in motor-vehicle traffic accidents by their activity at the time of accident, there were more drivers involved in accidents than there were persons in the other three activity classes combined. Of the total persons involved in accidents 61.3 percent were drivers, 36.8 percent were passengers, 1.6 percent were pedestrians, and 0.3 percent were others.

An interesting distribution, presented in figure 1, is the makeup of accidents of different severity. All persons in fatal injury accidents were not killed and all persons in nonfatal injury accidents were not injured. (For example, two cars, each with two occupants, may have collided. One occupant may have been fatally injured, another nonfatally injured, and the other two occupants may have avoided injury entirely.) Of the 718 persons involved in fatal injury accidents, 59 percent were killed, 23 percent were nonfatally injured, and 18 percent sustained no njuries whatever. Of the 93,634 persons nvolved in nonfatal injury accidents, 48 percent were injured and 52 percent were not njured.

In terms of number of accidents rather than nvolvement of persons, it may be noted from able 1 that of the 131,536 accidents reported 75 percent were property-damage-only accidents, and only 0.2 percent were fatal injury accidents.

Persons Injured and Injury Rates

In table 2 it is found that only 1 out of 8 persons involved in motor-vehicle accidents was injured and that more than half of those njured were not seriously injured. In terms of numbers, there were 421 persons fatally njured and 45,051 persons nonfatally injured. Of the persons nonfatally injured 27,006 experienced only superficial injuries such as mild shock, bumps, scratches, and bruises. The remaining 18,045 experienced serious njuries such as concussions, fractured bones, nternal injuries, deep cuts, and dismemberment of body extremities.

Data from table 2 also indicate that more han 1 out of every 3 persons killed in traffic accidents was a pedestrian. It is significant, too, that more passengers than drivers were killed in traffic accidents, since on the basis of average car occupancy of 1.6 persons, the relative exposure of drivers as compared to that of passengers was in a ratio of 5 to 3. As to the number of persons nonfatally injured n traffic accidents, drivers constituted approxmately one-half of the total.

In figure 2, the fatal injury rate—persons fatally injured in traffic accidents per 100,000 population—relates the high incidence of pedestrians and passengers in fatal accidents. Figure 3 shows the rate at which persons in traffic accidents were seriously and superficially injured. This latter chart points up the superficial nature of the majority of traffic accident injuries and also shows that a much higher percentage of drivers were seriously injured than any other activity group. Furthermore, it shows that most pedestrians not fatally injured were only superficially injured.

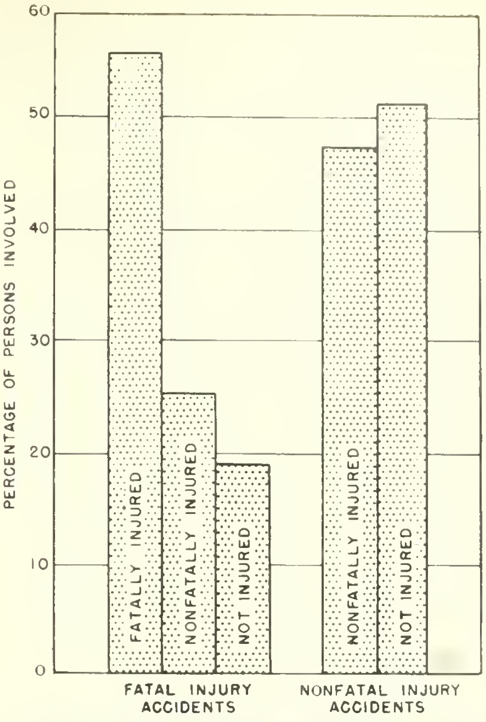

Figure 1.—Percentage distribution of persons involved in accidents, classified by severity of injury.

Age and sex of persons injured

Table 3 shows the number of persons and the number per 100,000 population in each age and sex group that were injured in traffic accidents, classified by severity of injury. Of every 100,000 persons in the State, 953 were injured in a traffic accident during the year and the majority of those injured were male persons. A total of 25,579 males were fatally and nonfatally injured, or 1,107 males per 100,000 male persons.

Categorized by age groups, more persons from 30 to 39 years of age were injured than in any other age group, and the next largest category was the 20-29 age group. Of all persons injured 43 percent were in these two age groups. However, of more importance from the economic point of view, 70 percent of the persons injured were from 20 to 59 years of age—the ages supplying most of the labor force of the State.

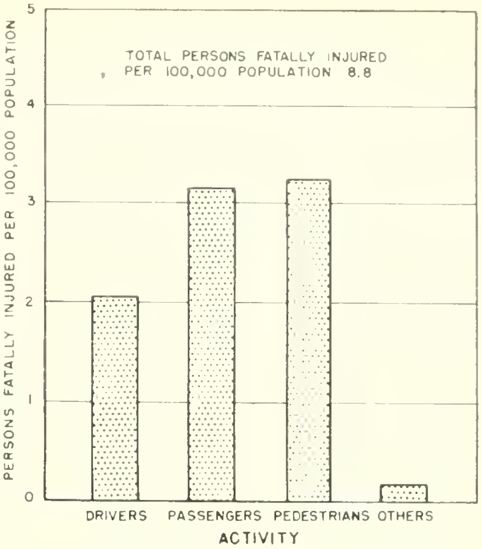

Figure 2.—Fatal injury rate, classified by activity.

In every age group except one, male persons had a higher fatal injury rate than females; and in the one exception the rate was the same for both sexes. In the 20-29 age group for males and in the 50 and over age groups for both males and females, a relatively high fatal injury rate occurred. The higher rates in the older age groups were believed to have been largely brought about by pedestrian deaths.

Turning to the nonfatal injuries per 100,000 in each age and sex group (table 3), males had a higher rate at every age, with only the youngest age group evidencing a closeness. Males in the 20–29 and 30–39 age groups had the highest injury rate, and females 60 and over had the lowest nonfatal injury rate.

Persons hospitalized

The number of persons nonfatally injured in traffic accidents that were hospitalized is shown in table 4. One of the most remarkable items derived from the 1953 Massachusetts accident study was evidence of the relatively

Table 2.—Number of persons in traffic accidents and number per 100,000 population, classified by severity and activity

Activity of persons	Persons injured					Total not injured	Total involved in accidents
	Fatally	Nonfatally			Total injured		
		Seriously	Superficially	Total			
Involvements:							
Drivers	102	12, 800	9, 434	22, 234	22, 336	199, 723	222, 059
Passengers	151	4, 275	12, 914	17, 189	17, 340	115, 895	133, 235
Pedestrians	158	770	3, 775	4, 545	4, 703	1, 190	5, 893
Others	10	200	883	1, 083	1, 093	---	1, 093
Total persons	421	18, 045	27, 006	45, 051	45, 472	316, 808	362, 280
Involvements per 100,000 population:							
Drivers	2. 1	268	198	466	468	4, 184	4, 652
Passengers	3. 2	89	271	360	363	2, 428	2, 791
Pedestrians	3. 3	16	79	95	99	25	124
Others	. 2	5	18	23	23	----------	23
Total persons	8. 8	378	566	944	953	6, 637	7, 590

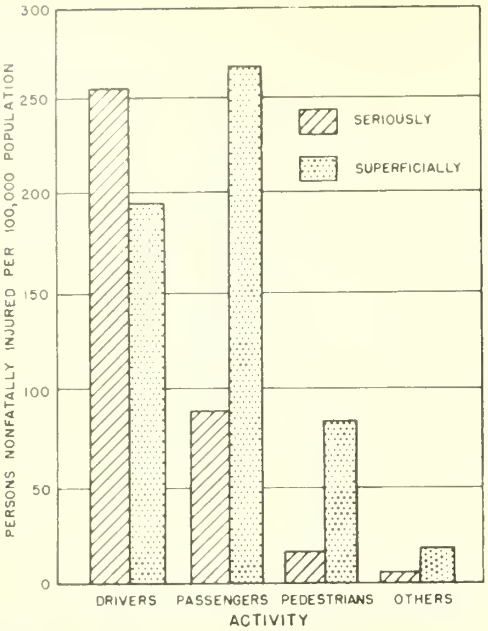

Figure 3.—Nonfatal injury rate, classified by activity and severity.

small number of nonfatally injured persons that required hospitalization. Only 15 percent of the persons nonfatally injured spent time in the hospital as a result of their injuries. Table 4 also shows that the 6,647 persons confined to the hospital remained there a total of 62,148 days or an average of 9.4 days per person.

During 1953, of the 45,051 persons nonfatally injured, 16,204, or 35.9 percent were examined in a hospital after the accident. Of those examined, 6,647 were confined while 9,557 persons were released from the hospital immediately after receiving emergency treatment. There were 28,847 persons, or 61.4 percent of all the persons nonfatally injured, that were either treated outside hospitals or received no professional treatment. These figures are additional evidence of the superficial nature of a large majority of traffic accident injuries.

Only 10 percent of the passengers that were injured required hospitalization, as compared with 16 percent of the drivers and 25 percent of the injured pedestrians. Persons in the "other" category that were hospitalized had the highest percentage rate, but the number of persons in this category represented less than 3 percent of the total number of persons injured.

Considered by activity class, over one-half of the persons requiring hospitalization were drivers, one-fourth were passengers, and one-sixth were pedestrians.

Table 4.—Number of persons nonfatally injured in accidents and time in hospital as the result of such injuries

Activity	Total persons nonfatally injured	Persons hospitalized	Days in hospital	Average days in hospital
Drivers	22, 234	3, 624	33, 804	9. 3
Passengers	17, 189	1, 630	15, 020	9. 2
Pedestrians	4, 545	1, 115	10, 580	9. 5
Others	1, 083	278	2, 744	9. 9
Total or average	45, 051	6, 647	62, 148	9. 4

Worktime lost

The number of nonfatally injured persons losing worktime and the amount of worktime lost as a result of motor-vehicle traffic accidents, shown in table 5, revealed the interesting point that more than one-half of the persons injured did not lose time from work. This situation appeared to be brought about by two factors: First, many injured persons were not members of the labor force, and second, three-fifths of the nonfatal injuries were of a superficial nature. The average number of work days lost per person was 19 days.

Of the 22,234 drivers nonfatally injured, 60 percent lost time from work, whereas a similar comparison for passengers and pedestrians show percentages of 28 and 23, respectively. Of the 19,405 persons losing worktime, drivers accounted for 68 percent; passengers, 25 percent; pedestrians, 6 percent; and others, 1 percent.

Persons Disabled

Table 6 shows the number of persons injured in traffic accidents that were disabled, and the disability rate per 100,000 population. As mentioned earlier in this article, the distribution made according to degree of disability was based on percentage data provided by Utah. On this basis, of the 45,051 persons nonfatally injured in traffic accidents in Massachusetts, 2,117 were permanently disabled to some degree, and 3,514 were temporarily disabled either totally or partially. It is important to note that of all those persons nonfatally injured only 225 suffered total permanent disability, or a rate of less than 5 persons per 100,000 population.

Table 3.—Number of persons injured in accidents and number injured per 100,000 population, classified by age, sex, and severity

Age of persons	Males injured			Females injured			All persons injured		
	Fatally	Nonfatally	Total	Fatally	Nonfatally	Total	Fatally	Nonfatally	Total
Number injured:									
Under 10	25	2, 993	3, 018	12	2, 238	2, 250	37	5, 231	5, 268
10–19	26	3, 070	3, 096	6	1, 694	1, 700	32	4, 764	4, 796
20–29	59	5, 187	5, 246	13	4, 542	4, 555	72	9, 729	9, 801
30–39	25	5, 579	5, 604	7	4, 297	4, 304	32	9, 876	9, 908
40–49	22	3, 668	3, 690	6	2, 741	2, 747	28	6, 409	6, 437
50–59	36	2, 958	2, 994	41	2, 776	2, 817	77	5, 734	5, 811
60 and over	75	1, 856	1, 931	68	1, 452	1, 520	143	3, 308	3, 451
Total	268	25, 311	25, 579	153	19, 740	19, 893	421	45, 051	45, 472
Number per 100,000 [1]									
Under 10	5	653	658	3	510	513	4	583	587
10–19	8	965	973	2	546	548	5	759	764
20–29	19	1, 673	1, 692	4	1, 402	1, 406	11	1, 535	1, 546
30–39	7	1, 631	1, 638	2	1, 161	1, 163	4	1, 387	1, 391
40–49	7	1, 207	1, 214	2	818	820	4	1, 003	1, 007
50–59	14	1, 160	1, 174	14	951	965	14	1, 048	1, 062
60 and over	23	575	598	17	369	386	20	462	482
Total	12	1, 095	1, 107	6	801	807	9	944	953

[1] Number of persons injured per 100,000 in each age and sex group.

Table 5.—Number of persons nonfatally injured in accidents and worktime lost from such injuries

Activity	Total persons nonfatally injured	Persons losing worktime	Total work-days lost	Average work-days lost per person
Drivers	22, 234	13, 261	271, 170	20. 4
Passengers	17, 189	4, 880	70, 795	14. 5
Pedestrians	4, 545	1, 053	23, 107	21. 9
Others	1, 083	211	4, 200	20. 0
Total or average	45, 051	19, 405	369, 272	19. 0

Table 6 also indicates the relative importance of temporary and permanent disabilities in the overall traffic accident injury picture. Numerically, disabilities are relatively small but from an economic point of view they account for a sizeable portion of every accident direct cost dollar. Considering the permanently disabled, only a small proportion had total disability. In contrast, almost 65 percent of the permanently disabled group were in the 25-percent disability group.

Total Direct Cost of Accidents

The total direct cost of motor-vehicle traffic accidents is summarized in table 7 by severity of accident and by cost elements In considering the direct cost of motor-vehicle accidents it is necessary to distinguish between property damage costs, which ma occur in any type of accident, and property-damage-only accidents. Property damag relates to an element of cost of an acciden whereas the property-damage-only acciden refers to the severity class of an accider where no injuries were sustained. Tot property damage costs of property-damag only accidents amounted to $17,900,000, whic

Table 6.—Number of persons nonfatally injured and the extent of their disability

Degree of disability	Number of persons nonfatally injured [1]	Persons nonfatally injured per 100,000 population
Permanent disability:		
Total	225	4. 7
75 percent	180	3. 8
50 percent	360	7. 5
25 percent	1, 352	28. 3
Subtotal	2, 117	44. 3
Temporary disability:		
Total	2, 435	51. 0
Partial	1, 079	22. 6
Subtotal	3, 514	73. 6
Total disabled	5, 631	118. 0
Total not disabled	39, 420	826. 0
Total injuries	45, 051	944. 0

[1] Distribution of persons according to degree of disabilit was derived from data provided by the State of Utah, wher a similar accident cost study was conducted.

Table 7.—Total direct cost of traffic accidents by cost elements and severity of accident

Cost elements	Severity of accident				All accidents
	Fatal injury	Nonfatal injury	Total injury	Property-damage-only	
	($1,000)	*($1,000)*	*($1,000)*	*($1,000)*	*($1,000)*
Property damage:					
Vehicle	238	9,352	9,590	17,722	27,312
Other property	2	102	104	204	308
Subtotal	240	9,454	9,694	17,926	27,620
Injury treatment:					
Doctors and dentists	25	2,038	2,063	-----	2,063
Hospital	24	1,117	1,141	-----	1,141
Ambulance	2	56	58	-----	58
Miscellaneous	3	139	142	-----	142
Subtotal	54	3,350	3,404	-----	3,404
Incidental costs:					
Loss of vehicle use	6	105	111	248	359
Value of time lost	32	4,061	4,093	473	4,566
Legal assistance and court fees	293	4,338	4,631	558	5,189
Damage awards and settlements	1,017	7,380	8,397	689	9,086
Subtotal	1,348	15,884	17,232	1,968	19,200
Total	1,642	28,688	30,330	19,894	50,224

as almost twice the property damage costs f total injury accidents. This, however, was o be expected because of the much greater umber of property-damage-only accidents. Vhen all the other elements of costs were conidered with the property damage costs, all ijury accidents cost 52 percent more than he property-damage-only accidents.

The average accident cost dollar, illustrated in figure 4 for each severity class and for each of the elements of cost, vividly describes the makeup of the $50,224,000 spent on accidents in Massachusetts during 1953. A most significant fact is that the cost of property damage was greater than that of all other elements of accident cost combined. The minor economic role of the treatment of injuries as a cost element is also indicated.

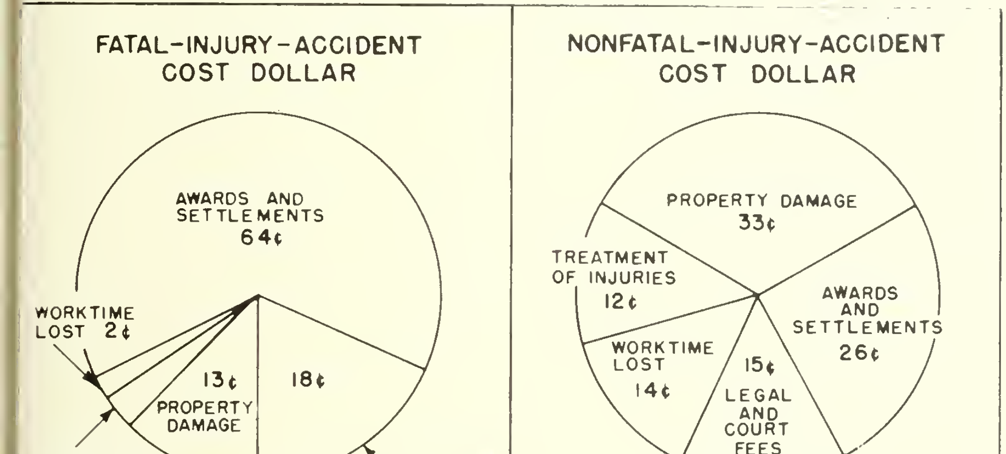

Figure 4.—Direct cost of traffic accidents shown as fractional parts of a dollar.

The "fatal injury accident dollar" is representative of the $1,642,000 spent on fatal accidents during 1953. It illustrates the overriding economic importance of damage awards and settlements, which in this instance largely represents the value of human life. The "fatal injury accident dollar" also shows that there is a fairly large element of property damage cost even in this most serious severity class of accident.

The "nonfatal injury accident dollar" represents the $28,688,000 spent on nonfatal injury accidents during the 12-month period. In the breakdown of this cost dollar the worktime loss cost more than the treatment of injuries, and property damage accounted for one-third of the total cost of nonfatal injury accidents. Also, legal and court fees (15 cents out of the dollar) cost more than half as much as awards and settlements (26 cents out of the dollar).

The "property-damage-only accident dollar" is representative of the $19,894,000 spent on property-damage-only accidents during the year. The significant thing about this diagram is that there were elements of cost other than property damage which accounted for 10 cents out of every property-damage-only accident dollar.

Table 8.—Direct cost of injuries by severity of injury and activity

Activity of persons injured	Fatally injured	Nonfatally injured [1]	All persons injured
	($1,000)	*($1,000)*	*($1,000)*
Drivers	156	11,552	11,708
Passengers	378	5,046	5,424
Pedestrians	824	2,305	3,129
Others	44	331	375
Total	1,402	19,234	20,636

[1] $13,906,000—cost of seriously injured; $5,328,000—cost of superficially injured.

Cost of injuries by activity

The direct cost to persons fatally and nonfatally injured in motor-vehicle traffic accidents is arrayed in table 8 by activity. The study did not provide a breakdown of costs for seriously and superficially injured persons according to their activity, but in total the amounts were $13,906,000 and $5,328,000, respectively. The $20,636,000 cost of injuries is the total direct cost of fatal and nonfatal accidents, less the cost of property damage incurred in them, as shown in table 7.

Over half of the total traffic accident injury cost was borne by drivers, who experienced approximately twice as much injury cost as passengers. Average injury costs per person involved are calculated to be $524 for drivers, $313 for passengers, $665 for pedestrians, and $343 for all other injured persons.

The average direct cost of injuries sustained by persons in traffic accidents, classified by the severity of injury, is shown in figure 5. The average cost of a fatal injury was $3,300, which was 4 times the amount expended for the serious nonfatal injury, and 17 times the cost for the superficial nonfatal injury. However, this comparison gives an erroneous idea of the economic importance of fatal injuries

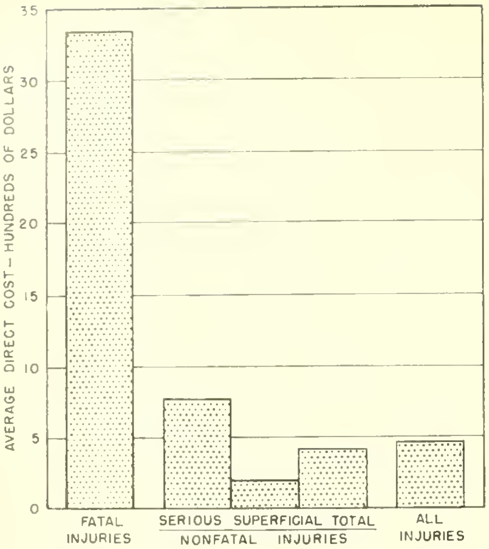

Figure 5.—Average direct cost of injuries sustained in accidents, according to severity of injury.

unless the total costs and number of persons are considered. The serious nonfatal injuries, in total, cost almost 10 times as much as fatal injuries. Referring back to table 2, there were 18,045 persons seriously injured, as compared to 421 persons fatally injured.

Cost of injuries by age and sex

Table 9 shows the direct cost of injuries to persons in accidents by the severity of their injuries and by their age and sex. It also shows the injury cost rate per 100,000 population for the different groups. Among the important facts observed was that nonfatal injuries, $403,000 per 100,000 persons, cost approximately 14 times as much as fatal injuries, $29,000 per 100,000 persons; and that the cost of injuries to males exceeded the cost of injuries to females by 11.4 percent. In the injuries of males, the nonfatal cost was 10 times as great as the cost of their fatal injuries; while in the injuries of females, the nonfatal injury cost was 23 times as great as the cost of their fatal injuries.

The per capita cost of fatal injuries resulting from motor-vehicle traffic accidents, calculated by dividing injury cost by the number of persons (total population) in each age and sex group, shows that the age groups of 20–29 and over 50 had the highest injury cost rates. The high cost rate in the 20–29 year group would appear to reflect the high injury rate and the greater exposure to accidents of this age group. The extremely high rates for persons over 50 years of age reflects two things: increased earning power that comes with age and experience, and failing physical characteristics that come with age.

The per capita costs of nonfatal injuries resulting from traffic accidents are shown in figure 6, calculated in exactly the same way used for calculating the per capita cost of fatal injuries. The cost of injuries to males generally increased with age—the 40–49 and the over 60 age groups being the only exceptions. The higher value of worktime lost and the slower rate of recovery in the 50–59 age group are clearly reflected in this chart.

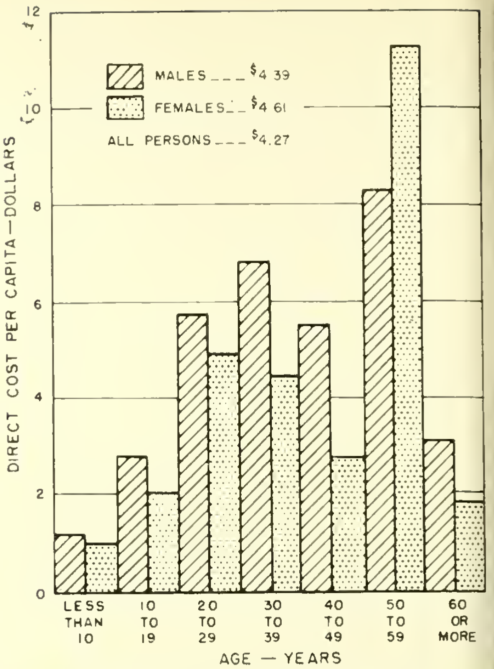

Figure 6.—Per capita direct cost, within each age and sex group, of nonfatal injuries.

Table 9.—Direct cost to persons injured in accidents and costs per 100,000 persons, classified by age, sex, and severity of injury

Age of persons	Males injured			Females injured			All persons injured		
	Fatally	Non-fatally	Total	Fatally	Non-fatally	Total	Fatally	Non-fatally	Total
Cost to persons injured:	*($1,000)*	*($1,000)*	*($1,000)*	*($1,000)*	*($1,000)*	*($1,000)*	*($1,000)*	*($1,000)*	*($1,000)*
Under 10	112	539	651	50	424	474	162	963	1,125
10–19	67	739	804	19	619	638	86	1,356	1,442
20–29	147	1,766	1,913	44	1,594	1,638	191	3,360	3,551
30–39	96	2,333	2,429	10	1,591	1,601	106	3,924	4,030
40–49	81	1,678	1,759	30	905	935	111	2,583	2,694
50–59	141	2,074	2,215	42	3,324	3,324	183	5,356	5,539
60 and over	370	1,009	1,379	193	683	876	563	1,692	2,255
Total	1,014	10,136	11,150	388	9,098	9,486	1,402	19,234	20,636
Cost to persons per 100,000:[1]									
Under 10	25	118	142	11	97	108	18	107	125
10–19	21	232	253	6	200	206	14	216	230
20–29	48	570	617	14	492	505	30	530	560
30–39	28	682	710	3	430	433	15	551	566
40–49	27	552	579	9	270	279	17	404	422
50–59	55	814	869	14	1,124	1,138	33	979	1,012
60 and over	115	312	427	49	174	223	79	236	315
Total	44	439	483	16	369	385	29	403	432

[1] Direct cost per 100,000 persons in each age and sex group.

The Economic Cost of Traffic Accidents in Relation to the Highway Systems

By **BERNARD B. TWOMBLY**,[1] **Traffic Engineer, Massachusetts Department of Public Works**

THE traffic accident toll in number of persons killed and injured has been iblicized so frequently that many people re quite familiar with the figures and can ven relate the details of the latest fatal ccident. Few persons, however, have an itimate knowledge of where, on a statewide asis, most of the accidents are occurring, hat types are most frequent, and what ould be the odds, as measured in number f accidents per 100 million vehicle-miles, of aving an accident on Massachusetts streets nd highways. Prior to the 1954–55 accident ost study conducted by the Massachusetts epartment of Public Works and the Registry f Motor Vehicles in cooperation with the .S. Bureau of Public Roads, the monetary ost of traffic accidents was purely conjectural.

The purpose of this article is to evaluate e efficiency of the highway systems in lassachusetts from the standpoint of losses rough automobile accidents. This article ows what the 1954–55 accident cost study vealed in terms of accident rates and direct sts of accidents on the State highways nd local roads in Massachusetts and also oints out where special attention should e directed in order to stem the tide of rising ccident toll. It is reasonable to expect that e conditions which prevail in Massachusetts re typical of those in many other States.

lighway Systems in Massachusetts

The highway network in Massachusetts n be classified into six major systems hose total extent in 1953 was 24,506 miles. he three general systems consist of the ederal-aid primary system, Federal-aid condary system, and non-Federal-aid (highays and roads not a part of any Federal-aid stem). Within each system two classificaons were made: State highways and local ads. A breakdown of total mileage and ehicle-miles traveled on each system during 953, the year upon which the accident cost udy was based, is shown in table 1.

ccident Experience on Highways and Roads

In 1953, there were 1.239 million registered assenger cars and 1.853 million licensed perators in Massachusetts. The vehicleiles driven in passenger cars during 1953 n the State's highways totalled 11.628 billion. In the same year, there were 131,536 passenger-car accidents which resulted in a direct cost of $50.224 million.

Massachusetts, though densely populated in many sections, still has a low proportion of urban areas when compared with rural areas. Approximately 15 percent of the 8,093 square miles are classed as urban and 85 percent as rural.

From table 2, the basic accident table, it is found that 88 percent of all accidents in the State occurred on urban highways and streets, which consisted of about one-fourth of the State's total mileage. This point is significant when considering density of accidents on the various highway systems. Moreover, two-thirds of all accidents in the State occurred in its 39 cities.

Table 2 also furnishes a comparison of the accident experience on the State highways and local roads. Without considering road mileage and traffic volumes, the 24,562 accidents on the State highways in comparison with the 106,974 accidents on the local road systems clearly show where the chief problem in accident reduction lies.

The Federal-aid primary and secondary systems accounted for 36 percent of the 131,536 accidents. Of the 47,681 accidents on these two systems, 24,080 occurred on State highways and 23,601 on local roads.

In terms of number of accidents classified by severity of accident, the property-damage-only accidents far outweighed the fatal and nonfatal injury accidents. Without the facts it is difficult to realize that 97,951 property-damage-only accidents occurred in one year and that these comprised 75 percent of all accidents. Local systems accounted for 80 percent of the property-damage-only accidents.

Accident experience rates

The average number of accidents per mile of road on State highways was 11.6 as compared with 4.7 on local roads. The mileage of the State highways was 2,109 miles while mileage of the local roads was 22,397 miles. Thus, the frequency of accidents on State highways was more than double that on local roads, but the State highways constituted only 9 percent of the road mileage in the State.

In the comparisons of motor-vehicle traffic accident rates the efficiencies of the various systems with respect to safety are made evident. Table 3 shows accident rates per 100 million vehicle-miles and is of value since it employs the widely accepted method of comparing accident experience on highways.

The accident rates in table 3 are most revealing. The figures show the following comparisons in accident rates on each system: On the Federal-aid primary system the accident rate (per 100 million vehicle-miles) on the State highways was 432, and 1,211 on local roads. On the Federal-aid secondary system the rates were reversed, with 1,537 accidents per 100 million vehicle-miles on the State highways and 399 on local roads. On the non-Federal-aid highways and roads, with a preponderance of mileage on the local roads, there were 225 accidents per 100 million vehicle-miles on the State highways and 2,107 on local roads.

The reversal of accident rates in the various systems occasioned special study in an at-

Table 1.—System mileage and vehicle-miles traveled by passenger cars on State highways and local roads in Massachusetts in 1953

Highway system	Rural		Urban		Total	
	Miles	Million vehicle-miles	Miles	Million vehicle-miles	Miles	Million vehicle-miles
Federal-aid primary:						
State highways	1,119	2,124	397	1,904	1,516	4,028
Local roads	119	180	333	1,253	452	1,433
Subtotal	1,238	2,304	730	3,157	1,968	5,461
Federal-aid secondary:						
State highways	359	303	79	131	438	434
Local roads	1,301	762	447	801	1,748	1,563
Subtotal	1,660	1,065	526	932	2,186	1,997
Non-Federal-aid:						
State highways	141	166	14	48	155	214
Local roads	14,987	2,350	5,210	1,606	20,197	3,956
Subtotal	15,128	2,516	5,224	1,654	20,352	4,170
All highways and roads:						
State highways	1,619	2,593	490	2,083	2,109	4,676
Local roads	16,407	3,292	5,990	3,660	22,397	6,952
Total	18,026	5,885	6,480	5,743	24,506	11,628

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[1] Presented at the 39th Annual Meeting of the Highway esearch Board, Washington, D.C., January 1960.

Table 2.—Motor-vehicle traffic accidents by highway system and severity for passenger cars registered in Massachusetts, 1953

Highway system	Number of accidents, classified by severity and location											
	Fatal			Nonfatal			Property-damage-only			Total		
	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total
Federal-aid primary:												
State highways	40	33	73	1,903	2,249	4.152	4.481	8.703	13.184	6,424	10,985	17,409
Local roads	-	49	49	15	5,216	5,231	------	12,081	12,081	15	17,346	17,361
Subtotal	40	82	122	1,918	7,465	9,383	4,481	20,784	25,265	6.439	28,331	34,770
Federal-aid secondary:												
State highways	24	5	29	576	410	986	3,257	2,399	5,656	3,857	2,814	6,671
Local roads	------	20	20	30	1,514	1,544	------	4,676	4,676	30	6,210	6,240
Subtotal	24	25	49	606	1,924	2,530	3,257	7,075	10,332	3.887	9,024	12,911
Non-Federal-aid:												
State highways	----	----	-	120	----	120	90	272	362	210	272	482
Local roads	26	118	144	1,304	19,933	21,237	4,676	57,316	61,992	6,066	77,367	83,373
Subtotal	26	118	144	1,424	19,933	21,357	4,766	57,588	62,354	6,216	77,639	83,855
All highways and roads:												
State highways	64	38	102	2,599	2,659	5,258	7,828	11,374	19,202	10,491	14,071	24,562
Local roads	26	187	213	1,349	26,663	28,012	4.676	74.073	78,749	6,051	100,923	106,974
Total	90	225	315	3,948	29,322	33,270	12,504	85,447	97,951	16,542	114,994	131,536

Table 3.—Motor-vehicle traffic accident rate by highway system and severity for passenger cars registered in Massachusetts, 1953

Highway system	Number of accidents per 100 million vehicle-miles, classified by severity and location											
	Fatal			Nonfatal			Property-damage-only			Total		
	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total
Federal-aid primary:												
State highways	1.88	1.73	1.81	89.59	118.11	103.07	210.96	457.09	327.30	302.40	576.94	432.17
Local roads	----	3.91	3.41	8.33	416.28	365.03	-------	964.16	843.65	8.33	1,384.35	1,211.51
Subtotal	1.73	2.59	2.23	83.24	236.45	171.81	194.48	658.34	462.64	279.42	897.40	636.67
Federal-aid secondary:												
State highways	7.92	3.81	6.68	190.09	312.97	227.18	1,074.91	1,831.29	1,303.22	1,273.26	2,148.09	1,537.32
Local roads	-	2.49	1.27	3.93	189.01	98.78	-------	583.77	299.16	3.93	775.28	399.23
Subtotal	2.25	2.68	2.45	56.90	206.43	126.69	305.82	759.12	517.37	365.07	968.24	646.56
Non-Federal-aid:												
State highways	----	--	--	72.28	------	56.07	54.21	566.66	169.15	126.50	566.66	225.23
Local roads	1.10	7.34	3.64	55.48	1,241.15	536.83	198.97	3,568.86	1,567.03	255.57	4,817.37	2,107.50
Subtotal	1.03	7.13	3.45	56.59	1,205.13	512.15	189.42	3,481.74	1,495.29	247.05	4,694.01	2,010.91
All highways and roads:												
State highways	2.46	1.82	2.18	100.23	127.65	112.44	301.88	546.03	410.65	404.58	675.51	525.27
Local roads	.78	5.10	3.06	40.97	728.49	402.93	142.04	2,023.85	1,132.75	183.80	2,757.45	1,538.75
Total	1.52	3.91	2.70	67.08	510.56	286.11	212.47	1,487.84	842.37	281.08	2,002.33	1,131.20

tempt to determine the reasons therefor. Comparisons of the number of accidents per mile of highway and the number of accidents per 100 million vehicle-miles were both studied in relation to their respective system mileages. In neither of these possibilities could a definite indication or trend be detected.

There did appear to be a relationship between the number of accidents in terms of vehicle-miles as related to their average traffic volumes. However, this was not sufficiently pronounced to substantiate any definite conclusions as to the reasons for the variation of accident rates in the various highway systems. A possible explanation might be that the priority given, since World War II, to the construction of controlled-access highways on the Federal-aid primary system, and the financial assistance to local governments for the improvement of local roads on the Federal-aid secondary system, have brought about a reduction in the accident rates on those systems.

In any event, the results of the accident cost study point to the need for improvement of local roads on the Federal-aid primary system and State highways on the Federal-aid secondary system, and even greater emphasis should be given to non-Federal-aid local roads with their high accident totals, extensive mileages, and high rate of accidents per 100 million vehicle-miles.

Total Direct Cost of Accidents

The foregoing has been purposely confined to a discussion of accident experience and accident rates on the highway systems. This was done in order to promote a clearer understanding of the accident picture in Massachusetts. It is a fact, however, that an important and integral part of any motor-vehicle accident, and aside from the aspect of personal injury, suffering and inconvenience involved, is the monetary cost incurred.

In this article, dollar values will be confined to direct costs only. The cost of accidents in relation to the highway systems is shown in tables 4 and 5. For those who include among their responsibilities the improvement of accident-prone highways, the overall costs of accidents on the highway systems in table 4 are of special significance. The value of table 5 is that it brings into sharp focus the cost of an accident to the individual or individuals involved. The cost varied from $203, for an average property-damage-only accident, up to $5,499 for an average fatal accident in urban areas.

In 1953, there occurred a total of 131,536 accidents which involved $50.224 million in direct costs. Fatal accidents made up 3 percent, nonfatal injury accidents 57 percent, and property-damage-only accidents 40 percent of the direct cost. In this cost breakdown it is interesting to examine the cost compariso in the light of what was previously foun through accident rate comparisons by agai using the highway systems. The costs an rates found below are derived from tables 3–

Federal-aid primary system

On the Federal-aid primary system, Sta highways had a better accident rate (per 1 million vehicle-miles of travel) than loc roads in the ratio of 1 to 3. However, t

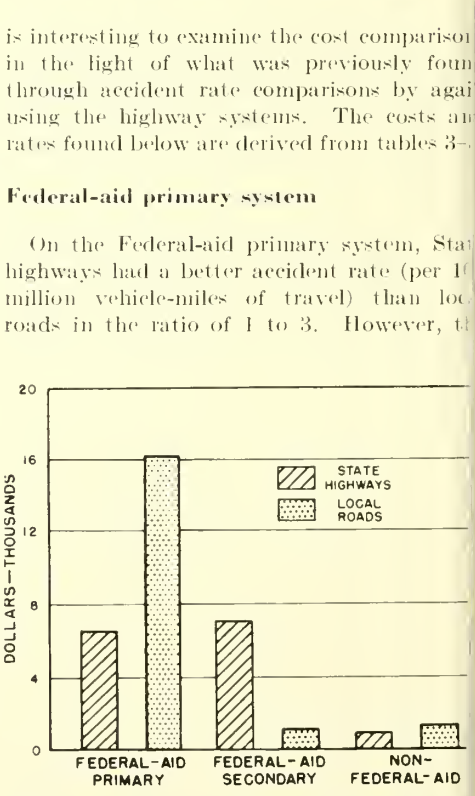

Figure 1.—Direct cost of traffic acciden per mile of highway in Massachusett classified by highway system.

tate highway accident costs exceeded those n the local roads, being $9.9 million and 7.3 million, respectively.

The highest total costs in each subdivision f the Federal-aid primary system were hose incurred in urban nonfatal accidents—hat is, in accidents where nonfatal injuries ere involved; they amounted to almost 8.9 million. State highways and local roads ach showed a total cost of approximately 4.4 million. Also, nonfatal injury accidents urban areas accounted for 52 percent of he total cost of all accidents on the Federal-id primary system. While property-damage-nly accidents were also very costly in total, both rural and urban areas, these costs ere only slightly more than half of the non-tal total costs.

ederal-aid secondary system

On Federal-aid secondary highways the tal accident cost was only 29 percent of e cost of accidents on the Federal-aid rimary highways. Here again, the dissimirity between accident rate comparisons and st comparisons is worthy of note. In the ederal-aid secondary system local roads had comparatively low accident rate and the tate highways experienced a much greater te, in the ratio of about 1 to 4. Neverthess, the accident costs were in the ratio of nly 2 to 3 on local roads versus State ighways.

Another noteworthy fact is that property-damage-only costs were greater on the secondary system than were those of fatal and nonfatal injury accidents combined.

Non-Federal-aid highways

The non-Federal-aid highways and roads form the large network of local streets and connecting highways in Massachusetts. Compared with the highways and roads in the Federal-aid systems, they carry far less average daily traffic. The accident rate for the non-Federal-aid highways and roads, expressed in number of accidents per mile of roadway, was less than the Federal-aid highways and roads and yet, they accounted for 69 percent of all accidents and 56 percent of the total accident cost in 1953.

Of the $28.0 million cost of accidents occurring on non-Federal-aid highways and roads, nonfatal injury accidents accounted for almost $16.0 million, property-damage-only accidents accounted for $11.3 million, and fatal accidents accounted for $751,460. Of the total amount, a very large proportion, $26.2 million, was incurred in urban areas. Thus, besides having a high accident rate in terms of accidents per 100 million vehicle-miles on its local road subdivision, accidents occurring on both non-Federal-aid State highways and local roads accounted for 56 percent of the cost of accidents in Massachusetts.

Economic Costs and Costs Rates

As previously noted, the efficiency of a highway in terms of accidents may be obtained by comparing accident rates. This is also true when comparing economic loss expressed in accident cost rates.

An analysis of figure 1, which shows the direct cost of accidents per mile of highway on each highway system, indicates that the highest accident cost per mile of highway, $16,112, was on the Federal-aid primary local roads. The second highest cost was $6,857 per mile on the Federal-aid secondary State highways and the third highest cost, $6,537 per mile, was on the primary State highways. Accident costs per mile on the remaining highways and roads were minor in comparison to the above three. It should be stressed, however, that these costs are average costs only and that costs fluctuate greatly on the many types of roadways in each system. Accident costs per mile, of course, present only one side of the picture and do not reflect the volumes of traffic carried.

The comparison of accident costs per passenger-car-mile, excluding trucks, is shown in figure 2. The cost in dollars per 100 million vehicle-miles has been reduced to fractions of a cent per mile. On the whole, the State highways are costing the motorist an average of 0.28 cent per passenger-car-mile while on the local roads the accident cost per passenger-car-mile is 0.53 cent.

Table 4.—Direct costs of traffic accidents by highway system and severity for passenger cars registered in Massachusetts, 1953

Highway system	Cost of accidents, classified by severity and location											
	Fatal			Nonfatal			Property-damage-only			Total		
	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total
Federal-aid primary:												
State highways	$214, 420	$179, 750	$394, 170	$1, 920, 530	$4, 428, 550	$6, 349, 080	$1, 189, 760	$1, 976, 520	$3, 166, 280	$3, 324, 710	$6, 584, 820	$9, 909, 530
Local roads		268, 640	268, 640	28, 800	4, 434, 680	4, 463, 480		2, 550, 290	2, 550, 290	28, 800	7, 253, 610	7, 282, 410
Subtotal	214, 420	448, 390	662, 810	1, 949, 330	8, 863, 230	10, 812, 560	1, 189, 760	4, 526, 810	5, 716, 570	3, 353, 510	13, 838, 430	17, 191, 940
Federal-aid secondary:												
State highways	83, 700	28, 820	111, 920	528, 500	416, 980	945, 480	655, 220	1, 290, 530	1, 945, 750	1, 267, 420	1, 735, 730	3, 003, 150
Local roads		115, 560	115, 560	4, 500	950, 410	954, 910		962, 920	962, 920	4, 500	2, 028, 890	2, 033, 390
Subtotal	83, 700	143, 780	227, 480	533, 000	1, 367, 390	1, 900, 390	655, 220	2, 253, 450	2, 908, 670	1, 271, 920	3, 764, 620	5, 036, 540
Non-Federal-aid:												
State highways				134, 400		134, 400	5, 430	19, 910	25, 340	139, 830	19, 910	159, 740
Local roads	106, 390	645, 070	751, 460	859, 200	14, 980, 900	15, 840, 100	695, 040	10, 548, 680	11, 243, 720	1, 660, 630	26, 174, 650	27, 835, 280
Subtotal	106, 390	645, 070	751, 460	993, 600	14, 980, 900	15, 974, 500	700, 470	10, 568, 590	11, 269, 060	1, 800, 460	26, 194, 560	27, 995, 020
All highways and roads:												
State highways	298, 120	207, 970	506, 090	2, 583, 430	4, 845, 530	7, 428, 960	1, 850, 410	3, 286, 960	5, 137, 370	4, 731, 960	8, 340, 460	13, 072, 420
Local roads	106, 390	1, 029, 270	1, 135, 660	892, 500	20, 365, 990	21, 258, 490	695, 040	14, 061, 890	14, 756, 930	1, 693, 930	35, 457, 150	37, 151, 080
Total	404, 510	1, 237, 240	1, 641, 750	3, 475, 930	25, 211, 520	28, 687, 450	2, 545, 450	17, 348, 850	19, 894, 300	6, 425, 890	43, 797, 610	50, 223, 500

Table 5.—Direct costs per traffic accident by highway system and severity for passenger cars registered in Massachusetts, 1953

Highway system	Cost per accident, classified by severity and location											
	Fatal			Nonfatal			Property-damage-only			Total		
	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total
Federal-aid primary:												
State highways	$5, 361	$5, 447	$5, 400	$1, 009	$1, 969	$1, 529	$266	$227	$240	$518	$599	$569
Local roads		5, 482	5, 482	1, 920	850	853		211	211	1, 920	418	419
Subtotal	5, 361	5, 468	5, 433	1, 016	1, 187	1, 152	266	218	226	521	488	494
Federal-aid secondary:												
State highways	3, 488	5, 644	3, 859	918	1, 017	959	201	538	344	329	617	450
Local roads		5, 778	5, 778	150	628	618		206	206	150	327	326
Subtotal	3, 488	5, 751	4, 642	880	711	751	201	319	282	327	417	390
Non-Federal-aid:												
State highways				1, 120		1, 120	60	73	70	666	73	331
Local roads	4, 092	5, 467	5, 218	659	752	746	149	184	181	276	338	334
Subtotal	4, 092	5, 467	5, 218	698	752	748	147	181	181	290	337	334
All highways and roads:												
State highways	4, 658	5, 473	4, 962	994	1, 822	1, 413	236	289	268	451	593	532
Local roads	4, 092	5, 504	5, 332	662	7, 639	759	149	190	187	280	351	347
Total	4, 495	5, 499	5, 212	880	860	862	204	203	203	388	381	382

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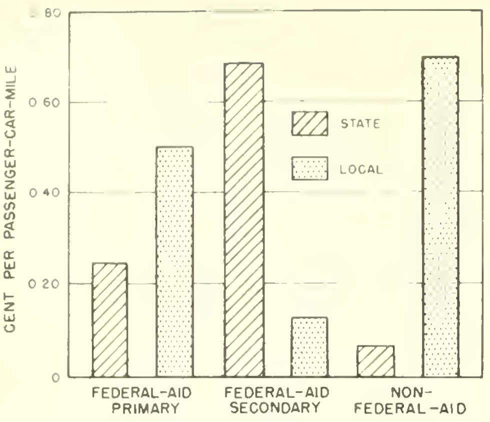

Figure 2.—Accident costs per passenger-car-mile on State highways and local roads in Massachusetts in hundredths of a cent.

Costs by type of accident

A breakdown of the number of accidents which occurred in Massachusetts in 1953 appears in table 6. Of the total accidents, 84 percent involved collisions of two or more motor vehicles, whereas 16 percent involved only one vehicle. In the former grouping, over 48 percent of the accidents were angle collisions. Rear-end collisions accounted for the next highest percentage, 20 percent, of the collisions with other vehicles. "Other" collisions, which included turning movements, parking maneuvers, and backing in traffic lanes, accounted for 12 percent of the collisions with other vehicles. Accidents involving only one motor vehicle included collisions with pedestrians, fixed objects, other objects (bicycles, scooters, etc.), and non-collision accidents.

By comparing the types of collisions on the State highways and local roads, it is interesting to note that rear-end collisions retained about the same proportion to the total collisions, 17 percent on State highways and 17 percent on local roads, but angle collisions were more prevalent on the local roads, 44 percent, whereas on State highways they accounted for only 23 percent of the accidents. Vehicle-pedestrian collisions accounted for only 2 percent of the collisions on State highways and 5 percent on local roads. Collisions with fixed objects were in about equal proportion, 6 percent on State highways and 5 percent on local roads.

The costs of the various types of accidents are shown in tables 7 and 8. Table 7 shows total costs and table 8 shows the accident cost rate per 100 million vehicle-miles of travel. The highest cost rate of accidents occurring on the State highways applied to head-on collisions. Rear-end and angle collisions had the next highest cost rates. On local roads the order of cost rates was angle collisions highest, rear-end second, and head-on collisions third.

Except for head-on collisions, sideswipe collisions involving two vehicles moving in opposite directions, single-car collisions with other objects, and non-collisions, the direct cost rates of accidents occurring on local roads were higher than the respective type of collision on State highways. In total costs, only the sideswipe (opposite direction) and non-collision accidents had a greater cost on State highways.

Table 6.—Motor-vehicle traffic accidents by type of accident and highway system for passenger cars registered in Massachusetts, 1953

Type of accident	Federal-aid primary		Federal-aid secondary		Non-Federal-aid		All highways and roads		
	State highways	Local roads	State highways	Local roads	State highways	Local roads	State highways	Local roads	Total
Two or more car collisions:									
Head-on	2,306	1,462	631	514	15	7,751	2,952	9,727	12,679
Rear-end	3,403	3,395	864	1,273	15	13,363	4,282	18,031	22,313
Angle	4,367	7,726	1,276	2,000	15	34,484	5,658	47,210	52,868
Sideswipe (same direction)	1,281	1,070	150	151	-----	4,397	1,431	5,618	7,049
Sideswipe (opposite direction)	433	226	15	90	-------	708	448	1,024	1,472
Others	2,129	1,333	1,086	1,650	181	10,570	3,396	9,953	13,349
Total	13,919	15,212	4,022	5,078	226	71,273	18,167	91,563	109,730
One-car collision with:									
Pedestrians	201	327	248	283	30	4,759	479	5,369	5,848
Fixed objects	745	734	679	726	15	4,361	1,439	5,821	7,260
Other objects	1,751	875	1,146	153	181	2,358	3,078	3,386	6,464
Non-collision accidents	792	213	577	------	30	622	1,399	835	2,234
Total	3,489	2,149	2,650	1,162	256	12,100	6,395	15,411	21,806
All accidents	17,408	17,361	6,672	6,240	482	83,373	24,562	106,974	131,536

In the Federal-aid primary system, reducing the cost rate of accidents per 100 million vehicle-miles to a cost rate per passenger-car-mile, the total cost for all types of accidents on the State highways was 0.24 cent per passenger-car-mile while the local road cost rate was 0.50 cent per passenger-car-mile.

The highest cost rate for all types of accidents occurring on the Federal-aid secondary highways and roads was the rear-end col-

Table 7.—Direct costs of traffic accidents by type of accident and highway system for passenger cars registered in Massachusetts, 1953

Type of accident	Federal-aid primary		Federal-aid secondary		Non-Federal-aid		All highways and roads		
	State highways	Local roads	State highways	Local roads	State highways	Local roads	State highways	Local roads	Total
Two or more car collisions:									
Head-on	$3,857,550	$958,730	$312,490	$288,060	$36,950	$3,513,630	$4,206,990	$4,760,420	$8,967,410
Rear-end	1,745,000	2,140,330	761,900	490,670	20,400	5,561,050	2,527,300	8,192,050	10,719,350
Angle	1,496,100	2,503,180	528,530	658,060	16,500	11,973,340	2,041,130	15,134,580	17,175,710
Sideswipe (same direction)	487,980	221,920	83,310	39,640	-----	1,104,170	571,290	1,365,730	1,937,020
Sideswipe (opposite direction)	339,850	97,100	43,500	33,040	-----	186,460	383,350	316,600	699,950
Others	452,570	293,600	465,170	186,600	5,430	912,600	923,170	1,392,800	2,315,970
Total	8,379,050	6,214,860	2,194,900	1,696,070	79,280	23,251,250	10,653,230	31,162,180	41,815,410
One-car collision with:									
Pedestrians	150,250	268,980	55,220	62,710	31,500	2,799,550	236,970	3,131,240	3,368,210
Fixed objects	547,850	670,290	225,310	145,710	35,400	1,395,380	808,560	2,211,380	3,019,940
Other objects	122,350	93,520	194,170	128,900	10,860	135,150	327,380	357,570	684,950
Non-collision accidents	710,030	34,760	333,550	-------	2,700	253,950	1,046,280	288,710	1,334,990
Total	1,530,480	1,067,550	808,250	337,320	80,460	4,584,030	2,419,190	5,988,900	8,408,090
All accidents	9,909,530	7,282,410	3,003,150	2,033,390	159,740	27,835,280	13,072,420	37,151,080	50,223,500

Table 8.—Direct costs of traffic accidents per 100 million vehicle-miles by highway system and type of accident for passenger cars registered in Massachusetts, 1953

Type of accident	Federal-aid primary		Federal-aid secondary		Non-Federal-aid		State highways	Local roads	Total
	State highways	Local roads	State highways	Local roads	State highways	Local roads			
Two or more car collisions									
Head-on	$95,768	$66,904	$72,002	$18,430	$17,266	$88,818	$89,970	$68,476	$77,119
Rear-end	43,322	149,360	175,553	31,393	9,533	140,573	54,048	117,837	92,186
Angle	37,142	174,681	121,781	42,102	7,710	302,663	43,651	217,701	147,710
Sideswipe (same direction)	12,115	15,486	19,196	2,536	-------	27,911	12,218	19,645	16,658
Sideswipe (opposite direction)	8,437	6,776	10,023	2,114	-------	4,713	8,198	4,554	6,020
Others	11,236	20,489	107,182	11,939	2,538	23,069	19,743	20,035	19,917
Total	208,020	433,696	505,737	108,514	37,047	587,747	227,828	448,248	359,610
One-car collision with									
Pedestrians	3,730	18,770	12,723	4,012	14,720	70,767	5,068	45,041	28,966
Fixed objects	13,601	46,775	51,915	9,322	16,542	35,273	17,292	31,809	25,971
Other objects	3,038	6,526	44,740	8,247	5,075	3,416	7,001	5,143	5,891
Non-collision accidents	17,627	2,426	76,855	-------	1,261	6,419	22,375	4,153	11,481
Total	37,996	74,497	186,233	21,581	37,598	115,875	51,736	86,146	72,309
All accidents	246,016	508,193	691,970	130,095	74,645	703,622	279,564	534,394	431,919

lisions on State highways. Angle collisions, which had the highest cost rate on the system's local roads, had the second highest rate on the State highways. "Other" collisions had the third highest rate on State highways while on the local roads head-on collisions were third.

Accident cost rates for all types of accidents on the secondary system's State highways greatly exceeded the rate of costs on the local roads. In terms of cost per passenger-car-mile, all costs in the Federal-aid secondary system on State highways were 0.69 cent as compared with 0.13 cent on the local roads.

By type of accident, the highest costs per 100 million vehicle-miles on State highways not on any Federal-aid system were the head-on collisions, collisions with fixed objects, and pedestrian accidents. On the local roads the angle collisions accounted for 43 percent of the total direct cost of accidents occurring on these roads. The cost rate amounted to $302,663 per 100 million vehicle-miles. This very high rate exceeded the rate for all other types of accidents occurring on any highway system by a wide margin. In comparing costs in each of the two highway classifications not a part of any Federal-aid system, it should be stressed that there were only 155 miles of State highways while there were 20,197 miles of local roads.

The cost per passenger-car-mile on the non-Federal-aid highways and roads was 0.07 cent on the State highways while the cost per passenger-car-mile on the local roads was 0.70 cent.

Costs on Selected Highways

In the analysis of the accident records and direct costs on the six highway systems, a general picture has been obtained on a Statewide basis. In order also to gain an impression of the costs on specific sections of highways it is of interest to consider the results of an accident cost study made in Massachusetts during 1957. This study was based on the accidents reported to the Registrar of Motor Vehicles in 1955. The purpose of the study was to determine the direct costs of accidents on two types of State highways; namely, those having no control of access and those on which access was controlled. Four highways were selected of the former type and they included a 2-, 3-, 4-, and 6-lane highway. Five highways of the latter type were selected and each were 4-lane divided highways which had been opened to traffic since June 1952. As part of the study, two curves, shown in figure 3, were prepared by plotting the yearly accident cost per mile against the average daily traffic volume for each of the nine sections of highway. The costs used were based on the results of the Massachusetts accident cost study and consisted of direct costs only. The results showed a yearly saving in cost of accidents per mile of highway of $18,000 at 10,000 vehicles per day and $68,000 at 25,000 vehicles per day on the controlled-access roads.

It is pointed out, however, that since this was an initial study, future research might alter the results obtained. Since this study was completed, a new comparison of accident costs on eight of the nine highways has been made available and is included here in table 9. The accident data used in this table were those submitted in 1957. A comparison of the results in accident costs per mile with the curves on figure 3 proved to be of special interest since volumes on the highways had changed since 1955. On one highway section, Route 20–2, the so-called Southwest Connection, volumes had decreased substantially because of the opening of the Massachusetts Turnpike. Despite this volume decrease, however, the yearly accident cost per mile was in close agreement with the curve in figure 3.

Table 9.—Accident rates and costs on State highways in Massachusetts comparing controlled-access highways and highways with no control of access, 1957

Sections of State highway	Miles studied	Average daily traffic	Severity of accidents				Accident rate [1]	Direct costs per mile
			Fatal	Nonfatal	Property-damage-only	Total		
Controlled access:								
Route 128 (Wellesley to Lynnfield)	24.6	32,200	3	254	211	468	161.7	$30,100
Route 15 (Sturbridge to Holland)	6.7	15,700	2	26	17	45	117.2	13,900
Route 2 (Concord to Westminster)	29.4	7,600	4	43	58	105	128.7	7,250
Route 1 (Danvers to Salisbury)	21.6	15,500	5	86	68	159	130.1	14,800
Total	82.3	[2] 17,700	14	409	354	777	146.2	------
Uncontrolled access:								
Route 1 (Revere to Peabody)	7.8	30,900	3	213	109	325	370.5	79,700
Route 20–1 (Northboro to Weston)	20.5	8,200	1	72	68	141	230.2	9,700
Route 20–2 (Northboro to Auburn)	15.9	8,600	2	122	82	206	412.7	21,600
Route 9 (Brookline to Framingham)	19.9	30,400	6	572	339	917	415.3	78,500
Total	64.1	[2] 17,900	12	979	598	1,589	378.4	-------

[1] Accidents per 100 million vehicle miles. [2] Weighted average.

On the other highways, accident costs per mile were also quite consistent with the curves with only one notable exception. In the volume range above 25,000 average daily

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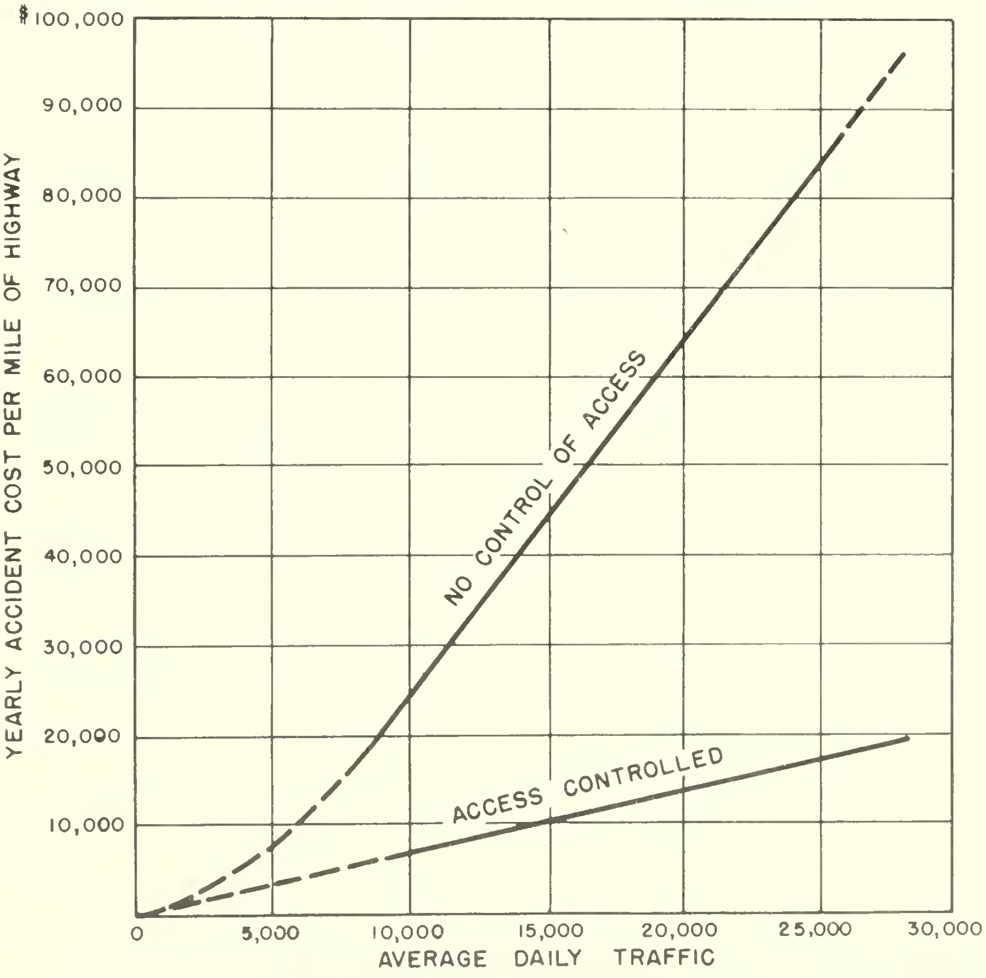

Figure 3.—Accident costs on Massachusetts highways with no control of access and with access controlled. Data based on 1955 accidents and traffic volumes.

The Economic Cost of Traffic Accidents in Relation to the Vehicle

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by [1] JAMES F. McCARTHY,
Transportation Economist

THIS article discusses traffic accidents in Massachusetts and their resulting economic costs. The study applies only to vehicles of Massachusetts registry that were involved in traffic accidents within the State. The data for passenger cars are for the year 1953, and for trucks for the year 1955.

The purpose of this article is to present the cost of accidents in relation to the vehicle in a way that will aid those interested in pinpointing where prevention efforts should be centered, as well as those engaged in the economic analysis of highway improvements. The intent is not to fix responsibility for accidents by type of vehicle but rather to show what kinds of vehicles were involved and the extent of involvement in terms of cost.

Involvement and cost data are analyzed separately for passenger cars and trucks by age of vehicle and severity of accident. Further analyses are made of trucks involved in accidents and their direct costs in relation to the registered gross weight and type of truck. Comparisons are made throughout the article on the basis of the involvement per 100 million vehicle-miles of travel and the accident cost per vehicle-mile.

An involvement is one vehicle in one accident. For example, in the passenger car study, if a passenger car struck a pedestrian there was one involvement in one accident. If two passenger cars collided, there were two involvements in one accident. Also, in the passenger car study, if a passenger car collided with a truck this was one passenger car involvement in one accident. The truck involvement was not included in the passenger car study.

Overall Accident Experience

There were 1,239,596 passenger cars registered in Massachusetts during 1953 and 179,610 trucks registered in 1955. Table 1 shows that passenger cars traveled 11.6 billion vehicle-miles as compared to 2.0 billion vehicle-miles for trucks, or a ratio of about 6 to 1. The direct cost of passenger car accidents was $50.2 million, whereas truck accident costs amounted to $4.8 million. Thus, the cost attributable to trucks was about one-tenth of that for passenger cars.

In addition to the direct costs of traffic accidents just mentioned, there were other direct costs in connection with non-traffic accidents, traffic incidents, and non-traffic incidents that are not included in this article. Non-traffic accident involvements—accidents involving motion but not occurring on a public roadway—resulted in direct costs of $2.7 million for passenger cars and $327,000 for trucks. Traffic and non-traffic incidents—mishaps not involving motion and usually involving vandalism, fire, or acts of God—resulted in direct costs of $4.7 million for passenger cars and $865,000 for trucks. The present analysis is confined to the $50.2 million for passenger cars and $4.8 million for trucks which comprised the direct costs of motor-vehicle traffic accidents.

A comparison is made in figure 1 of the direct cost of traffic accidents on the basis of vehicle-miles of travel for passenger cars and trucks. It is immediately evident that trucks had an appreciably lower accident cost per mile of travel. In the case of all accidents, the truck cost per mile of travel was 0.23 cent, and the comparable cost for passenger cars was 0.43 cent. For nonfatal injury and property-damage-only accidents, the passenger car cost per mile was approximately double that of trucks. Only for fatal injury accidents was the rate for trucks higher, being 0.014 cent for passenger cars and 0.027 cent for trucks for each mile of vehicle operation.

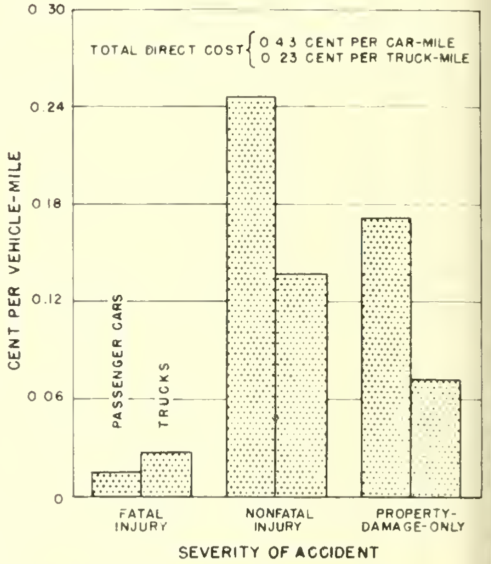

Figure 1.—Direct cost of traffic accidents in Massachusetts per passenger-car-mile of travel during 1953, and per truck-mile of travel during 1955, classified by severity of accident.

Table 1.—Number of motor vehicles registered in Massachusetts (passenger cars, 1953, and trucks, 1955) and their travel, classified by age of vehicle

	Age of passenger cars registered in 1953					Age of trucks registered in 1955				
	Under 2 years	2–4 years	4–8 years	8 years and over	Total	Under 2 years	2–4 years	4–8 years	8 years and over	Total
Number of vehicles	257,852	333,947	356,030	291,767	1,239,596	27,074	27,343	67,913	57,280	179,610
Percent of total	20.8	26.9	28.7	23.6	100.0	15.1	15.2	37.8	31.9	100.0
Vehicle travel:										
Total mileage (million miles)	3,331	3,390	3,143	1,764	11,628	355	396	795	495	2,041
Percent of total	28.6	29.2	27.0	15.2	100.0	17.4	19.4	39.0	24.2	100.0
Average annual travel (miles)	12,918	10,151	8,828	6,046	9,380	13,102	14,472	11,705	8,637	11,358

From the data given in tables 2 and 3, a comparison can be made of the average cost per involvement of passenger cars and trucks. The high cost per involvement in fatal injury accidents $4,800 for passenger cars and $6,800 for trucks—completely overshadows the cost per involvement of other types of accidents, but fatal injury accidents represented less than one-half of one percent of all accidents and only 3 percent of the total cost. The average cost for all involvements—$226 for passenger cars and $166 for trucks—was heavily influenced by the less costly property-damage-only involvements, which accounted for three-fourths of all accidents.

Age of Vehicles

The number of cars and trucks involved in accidents and the involvement rates by age of car and truck and severity of accident are summarized in table 2. For purposes of orientation on involvements and accidents,

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[1] Presented at the 39th Annual Meeting of the Highway Research Board, Washington, D.C., January 1960.

Table 2.—Number of cars and trucks involved in traffic accidents, and the rate per 100 million vehicle-miles, classified by age and severity

Age of motor vehicle	Motor vehicles involved in—								Number of motor vehicles per 100 million vehicle-miles of travel involved in—			
	Fatal injury accidents		Nonfatal injury accidents		Property-damage-only accidents		All accidents		Fatal injury accidents	Nonfatal injury accidents	Property-damage-only accidents	All accidents
	Number	Percent of total	Number	Percent of total	Number	Percent of total	Number	Percent of total				
Passenger cars:												
Under 2 years	66	19.2	10,403	19.2	31,162	18.6	41,631	18.7	2.0	312	936	1,250
2-4 years	94	27.3	15,311	28.2	47,375	28.3	62,780	28.3	2.8	452	1,397	1,852
4-8 years	101	29.4	16,762	30.9	51,927	31.0	68,790	31.0	3.2	533	1,652	2,189
8 years and over	83	24.1	11,784	21.7	36,991	22.1	48,858	22.0	4.7	668	2,097	2,770
Total	344	100.0	54,260	100.0	167,455	100.0	222,059	100.0	3.0	467	1,440	1,910
Trucks:												
Under 2 years	19	23.2	705	15.6	4,080	16.9	4,804	16.7	5.4	200	1,150	1,355
2-4 years	13	15.9	959	21.3	4,749	19.6	5,721	19.9	3.3	242	1,200	1,466
4-8 years	28	34.1	1,881	41.7	9,395	38.8	11,304	39.2	3.5	237	1,182	1,442
8 years and over	22	26.8	964	21.4	5,981	24.7	6,967	24.2	4.4	195	1,209	1,408
Total	82	100.0	4,509	100.0	24,205	100.0	28,796	100.0	4.0	223	1,186	1,412

22,059 passenger cars were involved in 31,536 accidents in 1953. This total was mprised of 344 involvements in 315 fatal ccidents, 54,260 involvements in 33,270 nontal injury accidents, and 167,455 involveents in 97,951 property-damage-only accients.

In contrast, 28,796 trucks were involved in 7,041 accidents during 1955. Classified by verity there were 82 truck involvements in 7 fatal accidents, 4,509 involvements in 344 nonfatal injury accidents, and 24,205 volvements in 22,648 property-damagenly accidents. The ratio of truck involveents to accidents was almost 1.1 to 1 28,796 to 27,041), whereas the corresponding atio for passenger cars involved in accidents as 1.7 to 1 (222,059 to 131,536).

ccident involvement rates

When the age of passenger cars and trucks volved in accidents was compared, dismilar results were found. The accident volvement rate of passenger cars, on the asis of involvements per 100 million vehicleiles, increased steadily with age—passenger rs 8 years and older had an involvement te of more than double that for cars under years of age. This influence of age on the volvement rates can be construed as a rong argument for vehicle inspection even ough there are contributory factors other an mechanical condition. With respect to ccident involvement rates of trucks, there was no apparent trend with age. Except for vehicles under 2 years of age, the involvement rate for passenger cars was higher than that for trucks. In the age group under 2 years, the truck rate exceeded the passenger car rate by only a narrow margin.

The influence of the age of the vehicle with respect to all accidents holds for each of the severity classes except for the fatal injury accidents. Throughout the article it will be seen that this class of accident severity has distinct characteristics in relation to nonfatal injury and property-damage-only accidents.

Just as for all accidents, the fatal involvement rate per 100 million vehicle-miles for passenger cars increased with age. When trucks under 2 years of age are ignored, the fatal involvement rate for trucks also increased with age. For vehicles less than 2 years of age, the fatal rate for trucks was almost 3 times that for passenger cars. For other than the newest vehicle group, the fatal involvement rates for passenger cars and trucks did not differ significantly.

Accident cost rates

The relationship between accident cost per million vehicle-miles and age of vehicle for each severity class of accident and for all accidents is shown in table 3 for passenger cars and trucks. A comparison of the costs for the two types of vehicles demonstrates the greater cost per mile of passenger car accidents. Although not as pronounced as the influence of age on accident involvement rates, the cost per million vehicle-miles for passenger cars tended to increase with age for all accidents, for the fatal injury accident, and for the nonfatal injury accident. The cost per million vehicle-miles for property-damage-only accidents dropped off for the older vehicles for both passenger cars and trucks. This undoubtedly reflected the reduced value of the older vehicles involved.

The fact that the total direct cost of nonfatal injury accidents constituted almost 60 percent of the total cost of all accidents for both passenger cars and trucks has a great influence on the pattern of cost per million vehicle-miles for all accidents.

Visual Classification of Trucks

It was not practical to classify passenger cars by different body styles or weight. However, trucks were studied first by visual classification and second by registered weight. In the visual classification, trucks were grouped as follows: 2-axle, 4-tire panel and pickup trucks; all other 2-axle, 4-tire trucks; 2-axle, 6-tire trucks; 3-axle trucks; and truck-tractor-semitrailer combinations. The truck-tractor-semitrailer combinations, ranging from 3- to 5-axle units, were treated as a single classification. Truck and full trailer combinations are illegal in Massachusetts and therefore were not represented in the study.

Table 3.—Total direct cost of traffic accidents involving passenger cars and trucks, and the accident cost per million vehicle-miles, classified by age and severity

Age of motor vehicle	Accident costs								Accident costs per million vehicle-miles of travel			
	Fatal injury		Nonfatal injury		Property-damage-only		All accidents		Fatal injury	Nonfatal injury	Property-damage-only	All accidents
	Amount	Percent of total	Amount	Percent of total	Amount	Percent of total	Amount	Percent of total				
Passenger cars:												
Under 2 years	$343,730	20.9	$5,572,710	19.4	$4,388,040	22.0	$10,304,480	20.5	$103	$1,673	$1,317	$3,093
2-4 years	507,170	30.9	8,695,640	30.3	7,264,870	36.5	16,467,680	32.8	150	2,565	2,143	4,858
4-8 years	462,820	28.2	8,015,350	28.0	6,001,250	30.2	14,479,420	28.8	147	2,550	1,910	4,607
8 years and over	328,030	20.0	6,403,750	22.3	2,240,140	11.3	8,971,920	17.9	186	3,631	1,270	5,087
Total	1,641,750	100.0	28,687,450	100.0	19,894,300	100.0	50,223,500	100.0	141	2,467	1,711	4,319
Trucks:												
Under 2 years	136,700	24.5	266,300	9.6	290,560	20.1	693,560	14.5	385	751	819	1,955
2-4 years	81,820	14.6	541,170	19.6	273,330	18.9	896,320	18.8	207	1,368	691	2,265
4-8 years	206,370	36.9	1,526,680	55.1	612,510	42.4	2,345,560	49.1	260	1,920	771	2,951
8 years and over	133,950	24.0	435,020	15.7	269,180	18.6	838,150	17.6	271	879	544	1,694
Total	558,840	100.0	2,769,170	100.0	1,145,580	100.0	4,773,590	100.0	274	1,357	709	2,340

2-axle trucks

Table 4 contains the travel characteristics and accident involvement rates for trucks. During 1955 there were 179,610 trucks registered in Massachusetts. The largest single group was the 2-axle, 4-tire panel and pickup trucks, which accounted for 43 percent of the total truck population and 37 percent of the total truck-miles of travel. Of almost equal magnitude were the 2-axle, 6-tire single-unit trucks. Together these two groupings constituted over 82 percent of the total number of trucks and 73 percent of the mileage traveled by trucks. The combination units, although representing less than 6 percent of the number of trucks, accounted for almost 17 percent of the truck-miles traveled.

The second group of trucks, other 2-axle, 4-tire trucks, had a disproportionate share of the accident involvements and direct costs, considering the number registered and miles traveled by this class. They generally had the highest involvement and cost rates (tables 4 and 5) of any truck group. There were 18,350 trucks of this group registered in Massachusetts during 1955. This number represented 10 percent of the truck registrations and 8 percent of the truck mileage. On the other hand, this group had 12 percent of the accident involvements, and 11 percent of the direct costs.

The accident involvement rate per 100 million miles of travel for the "other" 2-axle, 4-tire trucks was 8.4 for fatal injury, 238 for nonfatal injury, and 1,867 for property-damage-only accidents. The involvement rate for this group in all accidents was 2,114, which was 50 percent greater than the average rate of 1,412 for all trucks. The fatal injury involvement rate was more than 3 times that of 2-axle, 4-tire panel and pickup trucks.

Figure 2 compares the accident cost per mile of travel by truck group and severity. The accident cost per mile for "other" 2-axle, 4-tire trucks was 0.05 cent for fatal injury, 0.19 cent for nonfatal injury, and 0.07 cent for property-damage-only accidents or a total cost of 0.31 cent per mile. The cost rate for the fatal injury accidents was more than 3 times the cost rate for panels and pickups and twice the average cost rate of 0.03 cent for fatal injury accidents for all trucks. The nonfatal injury rate for other 2-axle, 4-tire trucks was 40 percent greater than the average nonfatal injury cost rate of 0.14 cent for all trucks. Only for property-damage-only accident involvements was the cost rate less than the average for all trucks. The total cost rate for this truck group was 32 percent greater than the average cost rate of 0.23 cent for all trucks. (The average cost rates for all trucks are shown in the totals in table 5.)

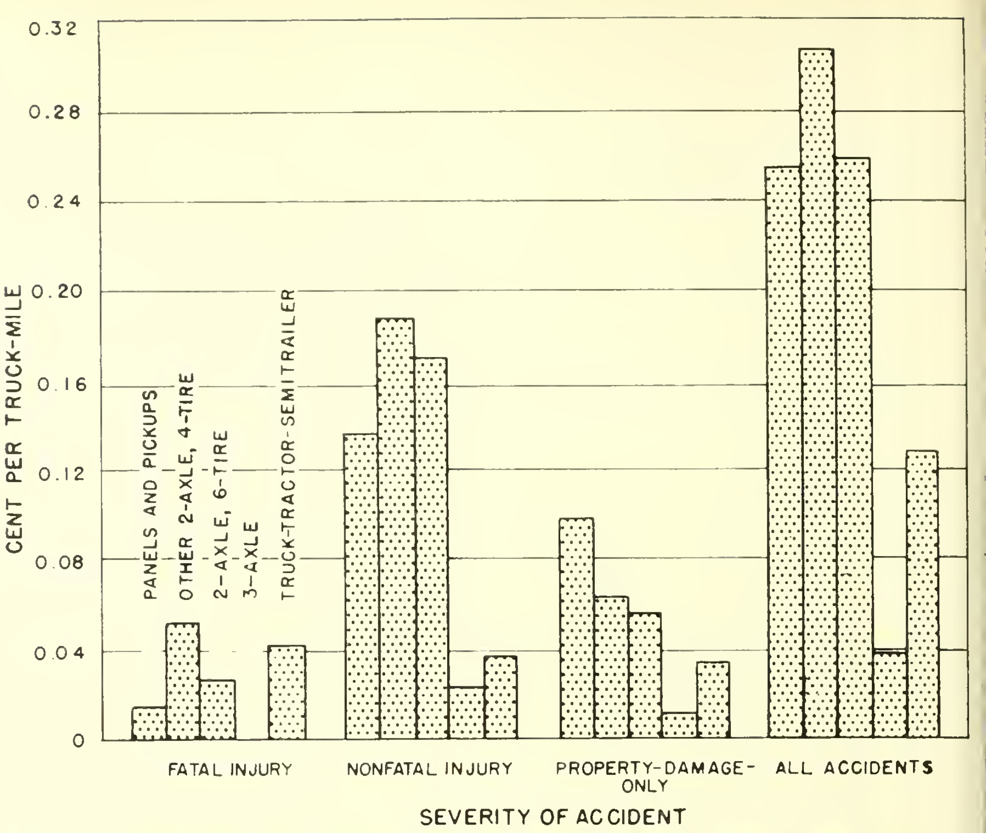

Figure 2.—Comparison of accident costs per mile of travel by visual classification of trucks and severity of accident.

Panels and pickups and the 2-axle, 6-tire trucks appeared generally to have about the same accident involvement rate and cost per mile. For all accidents, the involvement rates were 1,444 and 1,430, respectively. In figure 2 the accident cost per mile of travel is shown to be 0.255 cent for panels and pickups and 0.259 cent for 2-axle, 6-tire trucks; these two groups of vehicles which constituted 82 percent of the total registrations accounted for 80 percent of the total direct cost of truck accidents.

Considering the fatal injury accidents, the involvement rate per 100 million truck-miles for the panel and pickup trucks was 2.5 which was considerably less than the average rate of 4.0 for all trucks. The involvement rate for the 2-axle, 6-tire trucks in fatal accidents was 4.1 and was the third highest of the five vehicle types.

3-axle trucks

The fourth grouping of trucks, 3-axle trucks consisted of less than 2 percent of all trucks registered and accounted for the same percentage of the total number of accidents The total cost of accidents for this group was only 0.4 percent of the total cost of accidents for all trucks.

The number of 3-axle trucks involved in accidents per 100 million miles of travel was 992. Classified by severity, the involvemen rate was 1.9 for 3-axle trucks in fatal injury accidents, 86 in nonfatal injury accidents, and 904 in the property-damage-only accidents These involvement rates were, in general, the lowest rates for any of the five truck grouping

Table 4.—Number of trucks registered in Massachusetts during 1955, vehicle travel, involvement in accidents, and the accident involvemen rate per 100 million truck-miles, by visual classifications and severity

Truck groups	Number of trucks	Vehicle travel		Trucks involved in—								Accident involvement rate per 100 million truck-miles			
		Total mileage (thousands)	Average annual mileage	Fatal injury accidents		Nonfatal injury accidents		Property-damage-only accidents		All accidents		Fatal injury	Nonfatal injury	Property-damage-only	All accidents
				Number	Percent of total	Number	Percent of total	Number	Percent of total	Number	Percent of total				
Single-unit trucks:															
Panel and pickup	77,180	758,139	9,823	19	23.2	1,982	43.9	8,946	36.9	10,947	38.0	2.5	261	1,180	1,444
Other 2-axle, 4-tire	18,350	165,866	9,039	14	17.1	395	8.8	3,097	12.8	3,506	12.2	8.4	238	1,867	2,114
2-axle, 6-tire	70,670	727,412	10,293	30	36.6	1,683	37.3	8,691	35.9	10,404	36.1	4.1	231	1,195	1,430
3-axle	3,220	52,266	16,213	1	1.2	45	1.0	472	2.0	518	1.8	1.9	86	904	992
Truck-tractor-semitrailer	10,190	336,473	33,020	18	21.9	404	9.0	2,999	12.4	3,421	11.9	5.3	120	891	1,017
Total	179,610	2,040,096	11,358	82	100.0	4,509	100.0	24,205	100.0	28,796	100.0	4.0	221	1,186	1,412

Table 5.—Total direct cost of traffic accidents involving trucks and the accident cost per million truck-miles, by visual classification and severity

Truck group	Accident costs								Accident cost per million truck-miles of travel			
	Fatal injury		Nonfatal injury		Property-damage-only		All accidents		Fatal injury	Nonfatal injury	Property-damage-only	All accidents
	Amount	Percent of total	Amount	Percent of total	Amount	Percent of total	Amount	Percent of total				
Single-unit trucks:												
Panel and pickup	$114, 510	20. 5	$1, 050, 880	37. 9	$767, 860	53. 1	$1, 933, 250	40. 5	$151	$1, 386	$1, 013	$2, 550
Other 2-axle, 4-tire	88, 320	15. 8	315, 710	11. 4	107, 340	7. 4	511, 370	10. 7	532	1, 903	647	3, 083
2-axle, 6-tire	210, 260	37. 6	1, 256, 550	45. 4	414, 470	28. 7	1, 881, 280	39. 4	289	1. 727	570	2, 586
3-axle	--------	------	13, 410	0. 5	6, 360	0. 5	19. 770	0. 4	-----	257	122	379
Truck-tractor-semitrailer	145, 750	26. 1	132, 620	4. 8	149, 550	10. 3	427, 920	9. 0	433	394	444	1, 272
Total	558, 840	100. 0	2, 769, 170	100. 0	1, 445, 580	100. 0	4, 773, 590	100. 0	274	1. 357	709	2, 340

nder consideration. Similarly, the cost in ents per mile (fig. 2) was the lowest for any the five groups of trucks in all severity asses. These cent-per-mile rates were 0.03 ent for nonfatal injury, 0.01 cent for propty-damage-only, and 0.04 cent for all accients. There were no direct costs reported connection with the single reported involveent in a fatal accident of a 3-axle truck.

ruck combinations

The data for the fifth truck group, the uck-tractor-semitrailer combinations (hereafter referred to as combinations) offered a udy of contrasts. Although only 5.7 percent the total trucks registered, the combinations ere responsible for 12 percent of all traffic cidents involving trucks, and 9 percent of e total direct cost. On the other hand, the mbinations accounted for over 16 percent the total truck travel, and their rates of volvement and cost per mile for all classes severity except the fatal injury accidents ere below the average values for all trucks.

The breakdown of the cost of accidents for mbinations by severity class indicated that tal injury accidents accounted for 34 perent, nonfatal injury accidents 31 percent, and operty-damage-only accidents 35 percent of e total costs incurred. In contrast, condering the total direct cost for all truck accints, fatal injury accidents accounted for less an 12 percent of the total cost and nonfatal jury accidents and property-damage-only cidents accounted for 58 percent and 30 pernt, respectively.

Combinations had the second highest cost r fatal injury accidents. The average cost a fatal injury accident for this truck group was $8,100 as compared to an average cost of $6,450 for single-unit trucks. In addition to higher costs for fatal injury accidents, 1 out of every 190 combination involvements was fatal, whereas only 1 in every 409 involvements for single-unit trucks was fatal.

When viewed with consideration given to exposure in terms of truck-miles of travel, the higher percentages of truck combination involvements and costs are somewhat modified. The involvement rate for all accidents was 1,017 per 100 million miles of travel for combinations. Classified by severity this rate was 5.3 for fatal injury, 120 for nonfatal injury, and 891 for property-damage-only accident involvements per 100 million miles of travel. The cost of all accidents per mile of travel was 0.13 cent. By severity of accidents the cost rate was about the same, 0.04 cent for each class of severity.

From table 4 and figure 2 it appears that the truck combination involvement and cost rates were well below those of single-unit trucks for nonfatal injury and property-damage-only accidents. However, this was not so with regard to the fatal involvement and cost rates, as these rates for the truck combinations were considerably higher than the rates of single-unit trucks. The fatal involvement rate per 100 million miles was 5.3 for truck combinations, and 3.8 for all single-unit trucks. The cost of fatal accidents per mile of travel was 0.04 cent for the combinations compared to 0.02 cent for all single-unit trucks.

It becomes apparent from the foregoing discussion that any total view of trucks involved in accidents and their costs is predominately weighted by the number of panel and pickup trucks and 2-axle, 6-tire trucks, which dominated the total registrations, truck mileage, accident involvement, and cost rates. The combinations did play a rather important part in the total accident cost in relation to the number registered, but the large average annual mileage of this group tends to offset their disproportionate costs when viewed on an exposure basis.

Weight Classification of Trucks

The discussion up to this point has considered the relationship of truck and passenger car accident costs, and the variation of truck accident costs with age of vehicle and the visual classification of trucks. Another relationship of interest, particularly for economic analysis, is that which exists between accident costs and the weight classification of trucks—registered gross vehicle weights—as shown in table 6.

Of the 179,610 trucks registered during 1955 in Massachusetts, 56 percent had a gross vehicle weight under 8,500 pounds; 20 percent had weights between 8,500 and 16,500 pounds; 11 percent had weights between 16,500 and 24,500 pounds; 5 percent between 24,500 and 36,500 pounds; and 8 percent had gross weights of 36,500 pounds and over.

In addition to the registration data, table 6 indicates the truck travel for each registered gross vehicle weight group. It is important to note that the annual average mileage increased with registered weight. The increase between the last two weight groups is particularly significant. Trucks in the heaviest weight group traveled more miles in a year

able 6 .—Number of trucks registered in Massachusetts during 1955, vehicle travel, involvement in accidents, and the accident involvement rate per 100 million truck-miles, classified by gross weight and severity

ross vehicle weight (pounds)	Number of trucks	Vehicle travel		Trucks involved in—[1]								Accident involvement rate per 100 million truck-miles			
		Total mileage (thousands)	Average annual mileage	Fatal injury accidents		Nonfatal injury accidents		Property-damage-only accidents		All accidents		Fatal injury	Nonfatal injury	Property-damage-only	All accidents
				Number	Percent of total	Number	Percent of total	Number	Percent of total	Number	Percent of total				
Under 8,500	100, 940	949, 636	9, 408	41	50. 0	2, 662	59. 1	11. 001	45. 5	13, 704	47. 6	4. 3	280	1, 158	1, 443
8,500-16,500	36, 340	333, 960	9, 190	13	15. 8	722	16. 0	4. 998	20. 6	5, 733	19. 9	3. 9	216	1, 497	1, 717
16,500-24,500	19, 050	224, 139	11, 766	7	8. 5	416	9. 2	3. 289	13. 6	3, 712	12. 9	3. 1	186	1, 467	1, 656
24,500-36,500	9, 540	132, 603	13, 900	4	4. 9	245	5. 4	2. 084	8. 6	2, 333	8. 1	3. 0	185	1, 572	1, 759
36,500 and over	13, 740	399, 758	29, 094	13	15. 9	387	8. 6	2. 562	10. 6	2, 962	10. 3	3. 3	97	641	741
Total	179, 610	2, 040, 096	11, 358	82	100. 0	4, 509	100. 0	24, 205	100. 0	28, 796	100. 0	4. 0	221	1, 186	1, 412

There were 352 trucks involved in accidents of which the weights were unknown. These included: 4 trucks involved in fatal accidents; 77 in nonfatal accidents; and 271 in property-m age-only accidents.

Table 7.—Total direct cost of traffic accidents involving trucks and the accident cost per million truck-miles, classified by gross weight and severity

Gross vehicle weight (pounds)	Accident costs								Accident cost per million truck-miles of travel			
	Fatal injury		Nonfatal injury		Property-damage-only		All accidents		Fatal injury	Nonfatal injury	Property damage-only	All accidents
	Amount	Percent of total	Amount	Percent of total	Amount	Percent of total	Amount	Percent of total				
Under 8,500	$268,040	48.0	$1,915,460	69.1	$872,060	60.3	$3,055,560	64.0	$282	$2,017	$918	$3,218
8,500-16,500	130,990	23.4	378,530	13.7	242,910	16.8	752,430	15.8	392	1,133	727	2,253
16,500-24,500	19,990	3.6	209,950	7.6	126,920	8.8	356,860	7.5	89	937	566	1,592
24,500-36,500	46,900	8.4	71,600	2.6	73,110	5.0	191,610	4.0	354	540	551	1,445
36,500 and over	78,310	14.0	143,270	5.2	122,160	8.5	343,740	7.2	196	358	306	860
Unknown	14,610	2.6	50,360	1.8	8,420	0.6	73.390	1.5	------	------	------	------
Total	558,840	100.0	2,769,170	100.0	1,445,580	100.0	4,773,590	100.0	274	1.357	709	2,340

in total than those in any other group except those registered under 8,500 pounds.

Light trucks have often been compared to passenger cars because of many similar vehicle characteristics. One of these similarities is average annual travel per vehicle. The average annual travel of passenger cars and light trucks was approximately equal. Passenger cars averaged 9,380 miles of travel annually and light trucks averaged 9,408 miles. By comparing the involvement rate of 1,910 per 100 million vehicle-miles for passenger cars with the involvement rate of 1,443 for light trucks it may be seen that passenger cars were involved in 32 percent more accidents on a mileage basis.

Table 6 also contains the accident involvements and the involvement rate per 100 million miles of travel for each gross weight group. Table 7 contains the accident costs and cost rate per million truck-miles of travel for each weight group. When the proportion of heavy trucks involved in accidents is compared with the proportion of heavy trucks in the total truck registrations, the comparison presents a somewhat unfavorable picture for heavy trucks. For example, trucks with a gross vehicle weight of 36,500 pounds and over represented less than 8 percent of the total trucks registered in Massachusetts and over 10 percent of the trucks involved in accidents. In terms of accident costs, however, the heaviest trucks accounted for only 7 percent of the total accident costs.

Accident involvement rates

The involvement rates for fatal injury accidents appeared generally to decrease with registered weight over the range covered by the first three groups and then tended to level off. The proportion of trucks in the heaviest group involved in fatal injury accident was high in comparison to the proportion of trucks in the other groups. This group had one vehicle in fatal accidents for every 30 vehicles in nonfatal accidents. On an exposure basis, the heaviest truck group had a favorable involvement rate of 3.3 per 100 million miles of travel, as compared to a rate of 4.0 for the remainder of the trucks.

In the case of nonfatal injury accidents, the involvement rates decreased with registered weight. However, for the property-damage-only accidents the tendency was for an increase with weight excepting the heaviest group vehicles. For these trucks, 36,500 pounds and over, the rate dropped off sharply to less than half that for the other weight groups combined.

The involvement rate for all accidents, regardless of severity, was 741 for the heaviest trucks and 1,554 for all other trucks.

Accident cost rates

Accident cost rates per mile of travel are shown in figure 3. The heaviest trucks had the most favorable cost rates among the weight groups. For nonfatal injury accidents the cost rate of 0.04 cent per mile for the heaviest trucks was about one-fourth of the cost rate of 0.16 for all other trucks in acciden of comparable severity. The property-dan age-only cost rate was 0.03 cent per mile fo the heaviest trucks and 0.08 for all othe trucks. The cost rate for all accidents wa 0.09 cent per mile for the heaviest truck which was one-third of the cost rate of 0.2 cent for all other trucks.

A most significant finding of the report the very definite decrease of accident cost pe mile of travel with increase of registered gro vehicle weight. This is the first time suc data have become available. With referenc to the results for all accidents the cost ra ranges from 0.32 cent per mile for the lighte group to 0.09 cent per mile for the heavie group of trucks.

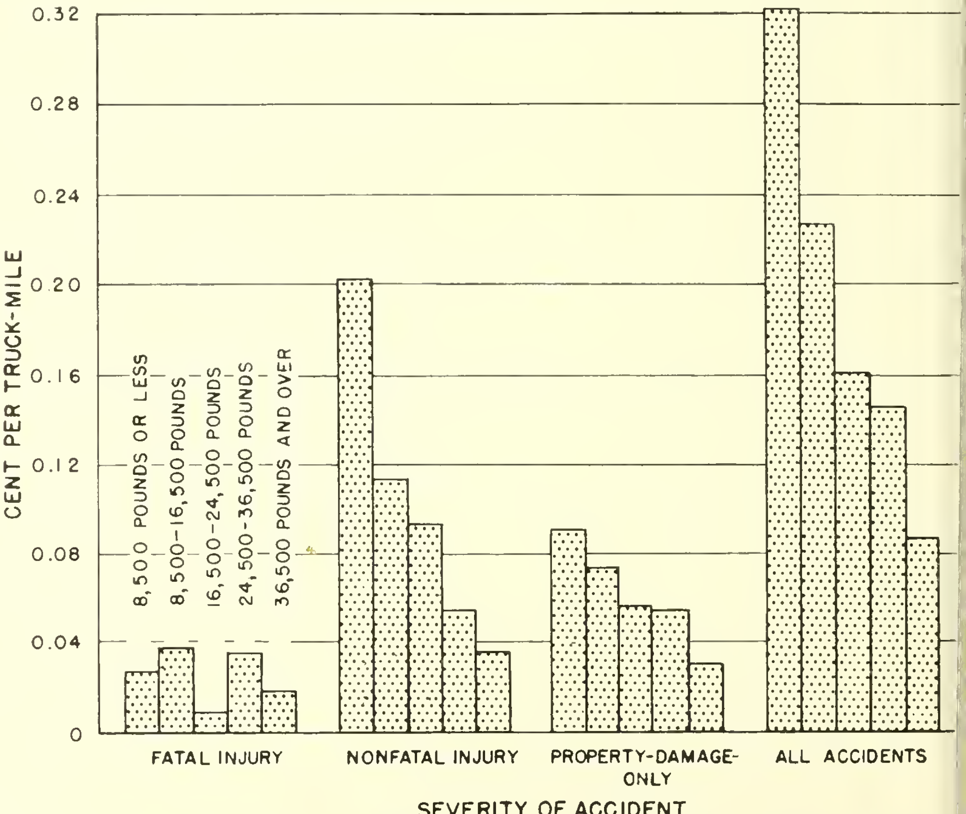

Figure 3.—Comparison of accident costs per mile of travel by gross vehicle weight an severity of accident.

The Economic Cost of Traffic Accidents in Relation to Highway Planning

and a Comparison of Accident Costs in Utah and Massachusetts

By J. E. JOHNSTON, Deputy Director of Highways for Planning, Utah State Deparment of Highways [1]

TODAY, highway planners are seeking more scientific methods of evaluating existing highways and designing and programing new facilities. The highway administrator and planner realize as never before the importance of engineering tools that will permit them to schedule construction on the basis of need, allocate highway funds on a priority basis, select additions to the highway systems, change system classifications, select proper alternate routes or locations, and to cope effectively with public and private groups having vested interests in the processes of planning and programing.

Some States have employed sufficiency ratings to assist them in accomplishing these objectives. This method simply assigns a point rating to each section of road according to its ability to provide traffic service in a safe and efficient manner. Other States have employed economic analyses using factors such as highway costs, revenues, and benefits. In the main, this approach has been simplified to include only the benefit quotient which reflects primarily a savings to the motor-vehicle user in operating cost and time through improved alinement. Some States use both approaches.

In the methods generally in use, one important factor is too often omitted—traffic accident rates. Some States incorporate accident rates in their sufficiency ratings, but more do not. It is known that sections of highway having a good adequacy rating, as provided by the sufficiency rating system, sometimes may have a high traffic accident frequency.

Traffic accident rates normally are not included in economic analyses of the cost-benefit variety at all. The principal reason for the omission has been the unreliability of traffic accident data. Too many accidents are not reported; and for those that are, the reports often do not clearly indicate where the accidents occurred. But one of the greatest deterrents has been the lack of information on accident costs related to the types of vehicles, classes of highways, roadway features, types of accidents, and severity of accidents.

Application of Accident Cost Factor

Traffic accident data should be one of the highway planner's most important tools to justify street and highway improvements. Just to illustrate this point, a traffic accident study was made on State Street in Salt Lake City. The study section, 17 blocks in length, carried over 30,000 vehicles daily. During a 3-year period, more than 1,000 traffic accidents occurred on this street, and over 700 of them took place at intersections. The street is a 6-lane divided roadway with parallel parking on each side.

It is estimated that had this traffic been carried on a street of freeway design, slightly more than 200 accidents would have been expected instead of 1,000. An Interstate System improvement is being planned to parallel State Street, and the portion paralleling the 17-block study section is expected to carry 100,000 vehicles daily by 1975.

According to the Utah accident cost study, the direct cost for passenger-car accidents occurring on major urban arteries was 0.49 cent per vehicle-mile. Using Utah study data, it was estimated that the direct cost of accidents on the Interstate System would be only 0.13 cent per freeway vehicle-mile. Thus, there would be a savings in accident costs of 0.36 cent per vehicle-mile on streets of freeway design. Based on an estimated traffic volume of 100,000 vehicles daily by 1975, the savings in accident costs would approach $330,000 annually.

The significant point is that an accident cost savings of such magnitude should not be overlooked in justifying an investment of $20 million, as would be required in constructing a freeway parallel to State Street. Furthermore, the savings figure is on the conservative side because it does not include indirect costs relating to traffic accidents.

In this analysis, the reduction in traffic accidents on freeways was based upon the California study [2] which indicated that there were five times fewer accidents on their freeways than on local streets. In addition, in the analysis, the accidents expected on the freeway were distributed as to type in accordance with those happening on the Detroit Expressway. This was done in order to isolate those types of accidents that could not occur on a freeway.

Utah and Massachusetts Results Compared

The Utah State Road Commission has completed a study of the passenger-car phase of traffic accident costs. It was based upon passenger cars registered by the Utah State Motor Vehicle License Bureau in 1955. A sample was selected from the registration list, a universe of 268,000 passenger cars. The registration sample was deemed to be of sufficient size to satisfactorily approximate the general accident experience of the motoring public, but in order to obtain more detailed information on fatal and nonfatal injury accidents the State's accident files were sampled at a much higher rate than the registration list.

Only a small portion of the data collected in the Utah accident cost study is reported here, in tables 1–4, to provide a comparison of the number of traffic accidents and their costs in Utah and Massachusetts.[3] When comparing the values for the two States, it must be kept in mind that the base year of the Utah study was 1955, and for the Massachusetts study it was 1953. Since only a two-year difference was involved, no attempt was made to adjust the data to a common year.

The size of Utah is 82,346 square miles, whereas in Massachusetts the area is 7,867 square miles. In 1955, Utah had a population of 797,000; there were 268,000 registered passenger cars, 455,000 licensed drivers, 31,400 miles of highways and streets, and 2,523 million vehicle-miles of travel. During 1953, there were 4,773,000 persons living in Massachusetts, 1,239,000 registered passenger cars, 1,858,000 licensed drivers, 24,500 miles of highways and streets, and 11,628 million vehicle-miles of travel.

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[1] Presented at the 39th annual meeting of the Highway Research Board, Washington, D.C., January 1960.

[2] *The Economy of Freeways*, by Lloyd Aldrich. City of Los Angeles, Street and Parkway Design Division, June 1953.

[3] A discussion of the Massachusetts data may be found in the article *The Economic Costs of Motor-Vehicle Accidents of Different Types*, by Robie Dunman, PUBLIC ROADS, vol. 30, No. 2, June 1958.

Table 1.—Comparison of the number of motor-vehicle traffic accidents involving passenger cars in Utah during 1955 and in Massachusetts during 1953

Item of comparison	Number of accidents		Percent of total		Number of residents per accident		Number of registered passenger cars per accident		Number of licensed operators per accident		Number of accidents per 100 million vehicle-miles of travel	
	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass.
Severity of accident:												
Fatal injury	77	315	0.2	0.2	10,350	15,152	3,481	3,933	5,909	5,898	3.1	2.7
Nonfatal injury	9,048	33,270	19.0	25.3	88	143	30	37	50	56	359	286
Property-damage-only	38,453	97,951	80.8	74.5	21	49	7	13	12	19	1,524	842
All accidents	47,578	131,536	100.0	100.0	17	36	6	9	10	14	1,886	1,131
Type of accident:												
Passenger-car collision with—												
Other motor vehicles	29,044	109,730	61.0	83.4	27	44	9	11	16	17	1,151	943
Pedestrians	1,792	5,848	3.8	4.5	445	809	150	210	254	315	71	51
Fixed objects	741	7,260	1.5	5.5	1,076	654	362	170	614	255	29	63
Other objects	11,921	6,464	25.1	4.9	67	746	22	194	38	290	473	55
Non-collision accidents	4,080	2,234	8.6	1.7	195	2,170	66	563	112	845	162	19
All accidents	47,578	131,536	100.0	100.0	17	36	6	9	10	14	1,886	1,131

Table 2.—Comparison of direct costs of motor-vehicle traffic accidents involving passenger cars in Utah during 1955 and in Massachuse during 1953

Item of comparison	Total direct cost		Percent of total		Total direct cost											
					Per accident		Per capita		Per passenger car registered		Per licensed operator		Per mile of road		Per 100 millio vehicle-mile of travel	
	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mas
Severity of accident:	(*$1,000*)	(*$1,000*)													(*$1,000*)	(*$1,0*
Fatal injury	284	1,642	1.2	3.3	$3,690	$5,213	$0.36	$0.34	$1.06	$1.32	$0.62	$0.88	$9	$67	11	14
Nonfatal injury	11,559	28,688	49.5	57.1	1,277	862	14.50	6.01	43.13	23.15	25.40	15.44	368	1,171	458	247
Property-damage-only	11,506	19,894	49.3	39.6	299	203	14.44	4.17	42.93	16.06	25.29	10.71	366	812	456	171
All accidents	23,349	50,224	100.0	100.0	491	382	29.30	10.52	87.12	40.53	51.31	27.03	743	2,050	925	43
Type of accident:																
Passenger-car collision with—																
Other motor vehicles	17,401	41,816	74.5	83.3	599	381	21.83	8.76	64.93	33.75	38.24	22.50	554	1,707	690	36
Pedestrians	1,417	3,375	6.1	6.7	791	572	1.78	.71	5.29	2.72	3.11	1.82	45	138	56	2
Fixed objects	271	3,023	1.2	6.0	366	414	.34	.63	1.01	2.44	.60	1.63	8	123	11	2
Other objects	1,880	673	8.0	1.3	158	105	2.36	.14	7.01	.54	4.13	.36	60	27	74	
Non-collision accidents	2,380	1,337	10.2	2.7	583	608	2.99	.28	8.88	1.08	5.23	.72	76	55	94	11
All accidents	23,349	50,224	100.0	100.0	491	382	29.30	10.52	87.12	40.53	51.31	27.03	743	2,050	925	43

In comparing the accident rates and costs for the two States, it is quite evident that the results were influenced by such factors as population density, travel speeds, urban characteristics, etc. It is likely that many of the differences could be explained, but to interpret the accident experience of States so widely separated geographically and having such dissimilar characteristics is beyond the scope of this article. Population density alone would be a major factor to consider in any analysis of traffic accidents. In Massachusetts there were 596 persons per square mile, as compared with 10 in Utah.

In spite of the dissimilarities of the two States, however, there is remarkable consistency in the relative distribution of accidents when classified according to severity and type. The most common method of comparison is to express the number of accidents and their costs in terms of 100 million vehicle-miles of travel. This is done in the last two columns of the tables. In general, accidents were costlier in Utah than in Massachusetts, regardless of severity or type of accident or type of collision.

An attempt to draw conclusions on the basis of the data presented here for two strikingly different States would not be justified, but certainly the results add significantly to the knowledge needed to develop national trends of traffic accidents.

Table 3.—Comparison of the number of collisions between passenger cars or passenger cars and other motor vehicles in Utah during 1955 and Massachusetts during 1953, classified by type of collision

Type of collision	Number of collisions		Percent of total		Number per 100 million vehicle-miles of travel	
	Utah	Mass.	Utah	Mass.	Utah	Mass.
Angle	9,911	53,320	34.1	48.6	393	458
Rear-end	10,580	22,501	36.4	20.5	419	193
Head-on	1,117	12,789	3.9	11.6	44	110
Sideswipe (same direction)	2,394	7,114	8.2	6.5	95	61
Sideswipe (opposite direction)	755	1,486	2.6	1.4	30	13
Turning movement	1,876	4,752	6.5	4.3	74	41
Parking maneuver and backing in traffic lane	2,411	7,768	8.3	7.1	96	67
All collisions	29,044	109,730	100.0	100.0	1,151	943

Table 4.—Comparison of direct costs of collisions between passenger cars or passenger c and other motor vehicles in Utah during 1955 and Massachusetts during 1953, classi by type of collision

Type of collision	Total direct cost		Percent of total		Average cost per accident		Cost per 100 million vehicle-miles of travel	
	Utah	Mass.	Utah	Mass.	Utah	Mass.	Utah	Mass
	(*$1,000*)	(*$1,000*)					(*$1,000*)	(*$1,00*
Angle	6,596	17,386	37.9	41.6	$666	$327	261	150
Rear-end	6,058	10,842	34.8	25.9	573	482	240	93
Head-on	1,834	9,078	10.5	21.7	1,642	709	73	78
Sideswipe (same direction)	1,157	1,958	6.7	4.7	483	276	46	17
Sideswipe (opposite direction)	351	706	2.0	1.7	465	471	14	6
Turning movement	486	1,114	2.8	2.7	259	232	19	10
Parking maneuver and backing in traffic lane	919	732	5.3	1.7	381	94	36	6
All collisions	17,401	41,816	100.0	100.0	599	381	690	360

Automatic Time Recorder for Dual-Drum Paver Operation

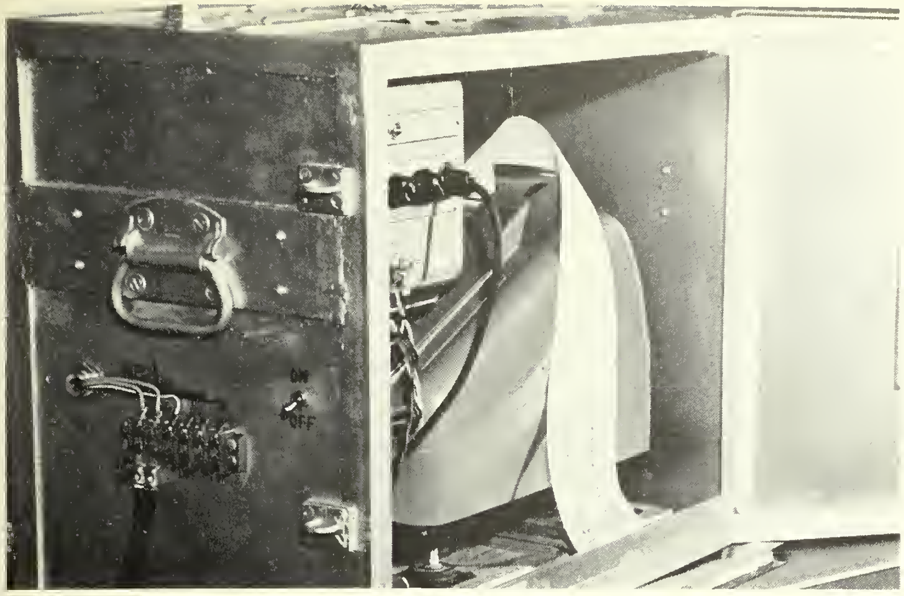


Figure 1.—Kilrec Timer (with panel door open).

ONE of the tedious tasks in research studies of portland cement concrete dual-drum paver operation is stop-watch timing of the paver cycle and mixing time. To do this job automatically, the Bureau of Public Roads recently developed a device which measures the elapsed time, in hundredths of a minute, of certain elements of the paver ycle, and prints the data on paper tape. The device, named the Kilrec Timer in recognition of its co-developers, M. J. Kilpatrick and V. N. Records, is assembled from commerially available components, including a printing timer, a suitable converter to provide 10-volt alternating current from the paver torage battery, and control circuits of relays, apacitors, and switches. The equipment is housed in a dustproof container with an xternal hookup panel. Intermittent demand or current is about 100 watts.

To operate the device, suitable switches, wired to the control mechanism, are attached o the transfer and discharge chutes of the paver. Closing of the transfer chute or pening of the discharge chute actuates the ppropriate switch, which completes an lectrical circuit and thereby causes the timer o record the time that has elapsed since the preceding switch actuation. The elapsed time nd an indication of which switch closed, is utomatically printed on a paper tape.

Figure 2 represents a section of tape, with interpreted data added on each side. On the ape itself, DT represents the closing of he transfer chute, and the time recorded pposite the symbol is the time interval from he opening of the discharge chute to the losing of the transfer chute. During this eriod only one batch is in the paver, in ither the first or second compartment of he drum.

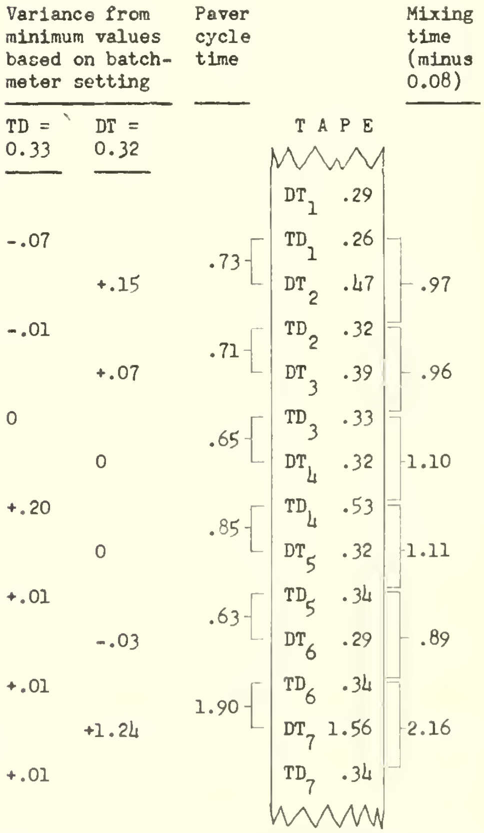
TD on the tape represents the opening of he discharge chute, and the time recorded pposite this symbol is the time interval from he closing of the transfer chute to the opening f the discharge chute. This is the period of imultaneous mixing, when there is a separate batch in each of the two compartments of the drum. (The printing timer actually prints the digits 1 or 2 on the tape, but DT and TD are used in the illustration, with successively numbered subscripts, for clarity in the explanation that follows.)

Referring to figure 2, it may be seen how actual paver cycle time is derived from the tape. The first simultaneous mixing interval, TD_1, was 0.26 minute. The next time interval, DT_2, was 0.47 minute, and accounts for discharging time, transfer time, and any delay time in raising the skip. The sum of these, shown at the left of the tape, was 0.73 minute, and this was the time elapsed during that particular paver cycle.

The data in figure 2 were obtained on a paver whose batchmeter was set for a minimum mixing time of 0.90 minute (54 seconds) and a minimum paver cylce of 0.65 minute (39 seconds). From these values, it was calculated that the minimum simultaneous mixing time, TD, should be 0.33 minute and the minimum nonsimultaneous mixing time, DT, should be 0.32 minute. In the two columns at the extreme left of the tape, in figure 2, comparison is made of the actual elapsed time, as recorded, with these minimum values. For example, TD_1, recorded on the tape as 0.26 minute, was 0.07 minute less than the minimum value of 0.33 minute. Minus signs indicate that the actual elapsed time was shorter than should be expected with the batchmeter setting; plus signs indicate longer elapsed time—that is, delays.

The total mixing time of each successive batch passing through the paver can also be derived from the data printed on the tape. The first batch recorded in figure 2, for example, entered the first compartment when the transfer chute closed, at the point in time represented by DT_1. It had been mixing 0.26 minute when the discharge chute opened (TD_1) to discharge the preceding batch from the second compartment. Mixing continued for the batch in the first compartment while discharge was underway. By the time the transfer chute had opened and closed again, shifting the batch from the first to the second compartment, at DT_2, another 0.47 minute had elapsed. When the discharge chute again opened, at TD_2, beginning discharge of the batch into the bucket, 0.32 minute more had elapsed.

The total elapsed time recorded for this batch was 0.26+0.47+0.32=1.05 minutes. However, from this total must be deducted the time lag between the moment of the first transfer chute closing and the entry of all solid material from the skip into the first compartment. This averages about 0.08 min-



Variance from minimum values based on batchmeter setting		Paver cycle time	TAPE	Mixing time (minus 0.08)
TD = 0.33	DT = 0.32			
			DT_1 .29	
-.07			TD_1 .26	
	+.15	.73	DT_2 .47	.97
-.01			TD_2 .32	
	+.07	.71	DT_3 .39	.96
0			TD_3 .33	
	0	.65	DT_4 .32	1.10
+.20			TD_4 .53	
	0	.85	DT_5 .32	1.11
+.01			TD_5 .34	
	-.03	.63	DT_6 .29	.89
+.01			TD_6 .34	
	+1.24	1.90	DT_7 1.56	2.16
+.01			TD_7 .34	

Figure 2.—A sample tape recorded by the Kilrec Timer, with interpreted data added.

ute. Hence the mixing time of this batch was $1.05-0.08=0.97$ minute, as shown at the right of the tape in figure 2.

From the illustration it can be seen that three batch mixing times, 0.97, 0.96, and 0.89 minute, were only a few seconds off the batchmeter set time. It will be noted, however, that there was considerable compensating fluctuation in the individual elapsed times of the three stages composing the total mixing time. The long mixing times of the third and fourth batches, 1.10 and 1.11 minutes, were due to the 0.20-minute delay in TD_4. In this step of the paver operation, the delay must have occurred in opening the discharge chute. The last batch illustrated had a mixing time of 2.16 minutes, due to the 1.24-minute delay in DT_7. This delay, which must have occurred in raising the skip, resulted in a mixing time almost 2½ times the batchmeter setting.

It is evident from this explanation of the data that can be derived from the Kilree Timer tape, that the device readily produces the following information:

1. The mixing time for individual batches of concrete.

2. The paver cycle time.

3. The frequency and duration of delays, and whether they occur at the skip or at the discharge chute.

4. The location of the individual batches, insofar as mixing time is concerned, in relation to other batches produced during the period that the timer is attached to the pave

While the Kilree Timer was developed f use in research, it could also readily be en ployed, either by the contractor or the co tracting authority's inspector, and either the job while the paver is operating or late for purposes of checking the operation of t paver and comparisons of actual with specifi mixing time.

Details as to the components and circuit used in building the Kilree Timer may obtained by writing to the Highway Nee and Economy Division, Bureau of Publ Roads, Washington 25, D.C. The cost the components and materials used in t recorder built by Public Roads was appro mately $900, of which $750 was for t printing timer.

The Economic Cost of Traffic Accidents in Relation to the Highway Systems

(*Continued from page 43*)

traffic, there was some variation from the 1955 curves. Values on the uncontrolled-access highways were somewhat lower than the curve in this volume range. One explanation for this may be that the over-crowding of the uncontrolled-access roads actually tends to prevent certain types of collisions.

Data for only one section of the controlled-access highways in this high volume range were available—Route 128, a section of the northern circumferential highway near Boston. This section of highway, although opened in 1952, has been reaching capacity and it is now being improved to higher standards. The accident rate on this highway section was 161.7 accidents per 100 million vehicle-miles in 1957. This was double the rate in 1953—accident costs per mile had increased from $19,100 in 1955 to $30,100 in 1957. When improvement of this section is complet there should be, without doubt, a reversal the upward trend.

In summation, it may be stated that t low accident rates and costs on the controlle access highways are consistent with previo findings. These highways most certain contribute their part in maintaining a co paratively low accident rate on the Sta highways in the Federal-aid primary syste

Highway Statistics, 1958

The Bureau's *Highway Statistics, 1958* is now available. The bulletin, the fourteenth of an annual series, presents the 1958 statistical and analytical tables of general interest on motor fuel, motor vehicles, highway-user taxation, State and local highway finance, highway mileage, and Federal aid for highways.

Included in the 1958 bulletin are motor-vehicle travel data for 1957 and 1958, data on contract bid prices for rural Federal-aid highway construction, and mileage data and status of the National System of Interstate Highways. In addition, there is included the status of the Highway Trust Fund, which gives the contributions and expenditures a current basis.

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U.S. Route 66, with frontage road construction, north of Joliet, Ill.

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Driver Tension Responses Generated on Urban Streets

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported [1] by RICHARD M. MICHAELS,
Research Psychologist

The purpose of this study was to relate river tension responses to events in traffic which caused the driver to make an overt change in speed or lateral location of the vehicle being driven. In order to measure the tension responses the galvanic skin reflex (GSR) was employed. Ten driver-ubjects were used on two urban test routes. nd, over a 2-week period, each subject rove each route 25 times. The routes were driven during five traffic time periods, ncluding peak, offpeak, and night.

The galvanic skin responses were measred continuously during each trip. The raffic events causing the responses were lso determined independently by an bserver and registered on the GSR record.

The results of the tests indicated that raffic events occurred at a rate of 1 every 1 to 35 seconds. Of these events, 85 percent generated a measurable galvanic skin esponse. It was also determined that of he measurable responses, 95 percent were aused by 8 types of traffic interferences: he most prevalent of which involved other vehicles in the traffic stream. This one ype of traffic interference accounted for ver 60 percent of all responses.

IN most analyses of the operational characteristics of traffic on an urban street, the ndividual motor vehicle is usually undifferntiated from the whole of traffic. Instead, he factors determining the operations of the treet are inferred from certain physical measrements of the mass of traffic itself. There ave been relatively few attempts to utilize river behavior itself as a measure of traffic haracteristics, or as a means for discriminatng among different streets (*1,2*).[2] In this tudy an attempt has been made to explore he possibility of using driver behavior as an nstrument for the analysis of highway and raffic characteristics.

In developing a measure of driver behavior rom which it would be possible to draw infernces about the highways under study, two lasses of questions were of particular interest:

- Are there stable characteristics of different treets that can be discriminated by some measure of driver behavior?
- Are there stable characteristics within a street and of the traffic on it which can be discriminated, using some measure of driver behavior?

The hypothesis which indicated the choice of behavioral measure was that the frequency and complexity of decisions required in driving on urban streets is so great that the driver is under a high level of tension. This hypothesis implies that the frequency and magnitude of tension responses aroused in driving will vary in some relation to the nature of the street and of the traffic.

Galvanic Skin Reflex

There are a variety of behavioral measures which may be employed to measure tension responses. However, for the purposes of this study it was desirable to have a measure that was directly relatable to events in traffic or the street on the one hand and also one that was a reliable indicator of driver tension response. A physiological measure that most nearly fulfilled these requirements is the galvanic skin reflex (GSR). This is a response occurring in the skin, manifesting itself as a change in the electrical resistance of the skin. The reflex is induced by activity of the autonomic nervous system and is initiated by unexpected stimuli that may be startling or tension inducing. The response appears as a decrease in skin resistance, and the magnitude of the reduction is correlated with the intensity of the inducing stimuli (*3*). Thus, the GSR represents one way to quantify the effects of an emotion-inducing stimulation.

An important characteristic of the GSR is its relation to the conscious experiences of the subject. It has been proved that there is a very high correlation between the GSR responses and the subject's awareness of the inducing stimuli (*4*)—a direct correspondence between the GSR itself and the event which caused its arousal. In general, the GSR is a means for quantifying those conscious experiences which arouse tension and, for the purposes of the study, this was ideal as it allowed a reliable relation of the emotional response to the traffic event which generated it.

Study Procedure

Two Washington, D.C., arterial streets served as test routes for this study. One route was a major arterial to and from the downtown Washington area. The second route paralleled and functioned as an alternate to the first route. Both were $4\frac{1}{2}$ miles in length. The characteristics of each were quite dissimilar. Route No. 1 served a considerable number of traffic and commercial land-use functions that were not found on the alternate route. The alternate route ran primarily through a residential area, had almost no commercial traffic, and transit facilities were located on only a small portion of its length. In addition, the street width varied from two to four lanes over the $4\frac{1}{2}$ miles. There was also considerable variation in both grade and curvature over this distance.

In order to relate galvanic skin responses to driving on the test routes, it was necessary to develop a list of types of traffic interferences since, in addition to the physiological measure of tension, it was necessary to specify the causes of these responses. This required establishing some criterion of what constituted an interference in driving.

Since the concern of the research was to relate street and traffic characteristics to driving behavior, it was decided that only those events which induced the driver to change either the speed or lateral location of the motor vehicle would be considered. This criterion implied that any change forced upon the driver was potentially tension inducing. An additional restriction imposed was that only the most direct cause of these changes would be classified as the inducing traffic event. Thus, for example, the driver might be in a stream of traffic which was forced to stop for a traffic signal. If the test vehicle were not the first to approach the signal, then the event which caused his change in speed was considered to be the vehicle in front of him rather than the traffic signal. Although such a distinction was arbitrary, it served two purposes. First, it reflected what the test driver directly responded to, even though he might have been aware of other events that were actually involved; and second, such a distinction greatly increased the reliability of observation.

This rationale oversimplifies the driving situation, for there was no question that the driver was aware of events considerably farther ahead than a car length. In addition, this system was limited in its ability to specify multiple and fast-changing situations.

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[1] This article was presented at the 39th Annual Meeting of he Highway Research Board, Washington, D.C., January 960.

[2] Italic numbers in parentheses refer to a list of references n page 71.

However, it was felt that reliability of observation was far more important than detailed specification of traffic interferences. From these considerations, a list of observed traffic events was developed. By code number, the observed tension inducing traffic interferences were:

1. *Parking maneuvers.*—Vehicles in the process of parking or already double parked.
2. *Marginal pedestrians.*—Pedestrians at or near the curb beginning to move into the street.
3. *Instream vehicles.*—Vehicles in the traffic stream in front of or adjacent to the test vehicle.
4. *Transit loading platforms* (route No. 1).—Loading platforms located in the street. *Opposing vehicles* (route No. 2).—Vehicles approaching the test vehicle from the opposite direction.
5. *Instream pedestrians.*—Pedestrians already in the street either directly in or approaching the path of the test vehicle.
6. *Diverging vehicles.*—Vehicles attempting to leave or disassociate from the traffic stream.
7. *Merging and crossing vehicles.*—Vehicles other than those in the stream of travel attempting to join or cross the traffic stream.
8. *Traffic signals.*—Traffic controls or signals which directly induced a change in the speed of the test vehicle.

The eight broad categories were determined in part by the nature of the street. Thus, on route No. 1, streetcar loading platforms were obvious interferences and were specifically included. On route No. 2, however, this was replaced by a medial friction event. In addition to being based on the actual nature of the study routes, these events were also predicated upon certain friction concepts of traffic flow. As may be seen, the list is a compromise between the general case of four frictions and the more specific individual conflicts that occur in traffic.

For the conduct of the experiment, a passenger car with automatic transmission was used. Study teams consisted of a driver an observer, and a recording instrument operator. The observer served as the team leader during the experiment. It was the duty of the observer to specify when a traffic event occurred and which event caused the change in vehicle operation. These were reported to the instrument operator who was seated in the rear seat of the vehicle. He in turn coded the information on the GSR recorder, and was the only member of the team to have knowledge of the driver's responses.

The drivers were instructed to travel the route by floating with the traffic wherever possible. With these instructions, the general pattern of driving was quite consistent among the test drivers. Although some differences in driving patterns and driving habits were noted, the differences were generally quite minor and ordinarily very subtle.

At the outset of the test run, electrodes were placed on the first and third fingers of the left hand of the test driver. Although a variety of electrode placements were tried, it was found that placement on these two fingers gave the most sensitive response, were mechanically the simplest to place, and allowed the driver to operate the vehicle normally. After placement of the electrodes, a normal resting level (that is, a basal level) for skin resistance was determined. The recording machine contained a feedback circuit for balancing out drift in the basal level. Thus, only deviations from an arbitrary zero level were measured.

The sensitivity level for each driver was adjusted empirically at the beginning of the test run. Usually a startle stimulus was employed and the sensitivity raised until the full scale of deflection was obtained. For all drivers the required sensitivity range was quite narrow. Determining the sensitivity range did not, of course, eliminate individual differences among the drivers, but it did reduce the variability for an individual driver from run to run.

Two phases of study

The study was conducted in two phases. First, each of the five test drivers drove both routes in one continuous cycle, and the differences between the two routes were compared. For these runs four complete cycles were obtained for each driver during the offpeak traffic hours.

In the second phase, each route was studied individually during five workday traffic periods. The runs were made during the morning peak hour traffic, morning offpeak traffic, afternoon offpeak traffic, afternoon peak hour traffic, and nighttime traffic. Each of the five drivers drove a complete cycle of each route three times during the peak hours and twice during the offpeak hours. The design was to have each driver observed an equal number of times by the two different observers.

In either phase of the study, at the beginning of the run, the time was noted and the drivers floated with the traffic from one end of the route to the other. The observer reported to the instrument operator any changes meeting the criteria of traffic interference and what events caused the change. At the end of the route there was a 10-minute rest durin which time the basal level of skin resistanc was again determined. Usually there was slight increase of level from the beginning t end of the route, especially on the first ru After the rest period, the driver proceeded t complete the full cycle.

Data recorded

Observers were instructed to report event to the instrument operator as quickly as pos sible in order to minimize errors in locatin occurrences in time on the GSR record. sample record of the galvanic skin response figure 1, shows that a notation was generall placed simultaneously with or slightly preced ing the response. Whenever an observatio preceded a response by more than 5 second the two were considered unrelated. Similarl an observation that followed a response b more than 1 second was considered unrelate to the response. Both these criteria wer predicated upon the assumption that the la in the GSR response was longer than for th verbal response. In practice, uncertainty c association of observation and response oc curred in no more than 5 percent of th occurrences.

As may be seen from figure 1, some response were not associated with traffic events, suc as the small peak prior to the first event (even No. 7), and were not included in the analysi The GSR is sensitive to a variety of events, in ternal as well as external, so that unidentifiabl responses were fairly common. Also, a GS did not occur every time there was a traffi event, as exemplified in the case of the las event (event No. 3). Nevertheless, thes were included in the data analysis. It wa found that approximately 15 percent of th traffic events observed aroused no respons on the part of the driver.

Magnitude of GSR

The meaning of the magnitude of the gal vanic skin response needs some explanatio The recorder actually measures the GSR i

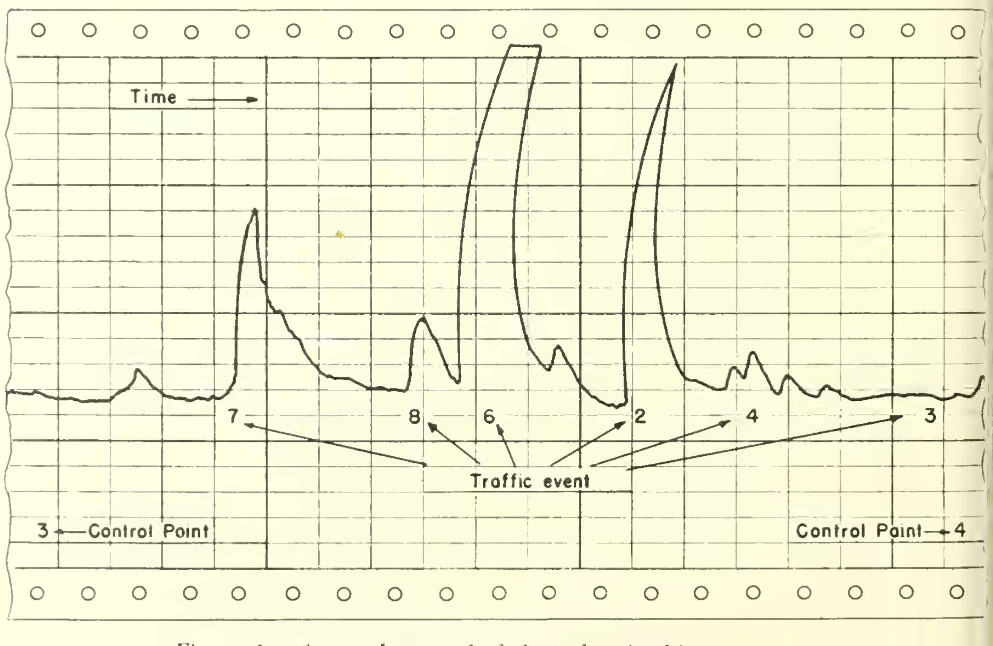

Figure 1.—A sample record of the galvanic skin responses.

nits of log conductance. This unit is em-loyed because the magnitude of change in kin resistance is more nearly linearly related o the magnitude of the inducing stimulus vhen resistance is measured in log conductance 5). Thus, the successive vertical divisions n the recorder chart represented equal incre-nents of GSR. However, the absolute values f these divisions were not known, for these lepended upon the setting of the sensitivity cale. With the method of calibration used, he values for each scale division were essen-ially relative to the base level. They are, as consequence, unitless, and may be termed reaction units.''

able 1.—Comparison of tension responses recorded in the continuous test run over both routes, made during offpeak traffic periods

	Route No. 1	Route No. 2
Total events	445	237
Events per minute	2.42	1.71
Average GSR magnitude	2.28	2.23
Magnitude response per minute	5.52	3.81

Test Results

The data were tabulated for each driver for ach run and each route, and were broken lown by the observed traffic events and the nagnitude of response associated with each vent. In addition, the travel time for each rip was recorded. It was possible from these lata to determine the average magnitude of esponse for each event, and with the time neasures to determine both the rate at which vents occurred and the magnitude of re-ponses that occurred in time.

Analyzing the data recorded during the con-inuous test run over both routes, study phase No. 1, table 1 contains the frequency of occur-nce of events and the average magnitude of esponse for each event. In this tabulation he data for all drivers were combined. The lifference in the total number of events in art reflects the difference in travel time for he two routes. In order to better equate the outes, a rate figure was computed and is shown in the table as the events per minute. Here it may be seen that the difference be-ween the two routes are reduced somewhat. Nevertheless, there were still 42 percent more responses per unit time on route No. 1 than on the alternate route. On route No. 1 there was a traffic event once every 24.7 seconds while on route No. 2 there was an event every 34.9 seconds. Since 85 percent of the events aroused the GSR, there was, consequently, a tension inducing event every 29.2 seconds on route No. 1 and 41.4 seconds on the alternate. It is quite evident from these data, then, that drivers on the alternate route faced considerably fewer interferences with driving than on the major arterial route.

These results reflect only the frequency of occurrence of events and do not directly relate to the magnitude of the galvanic skin responses. The average magnitude of response alone was inadequate since it did not reflect the differences in travel time for the two routes. One simple measure is the total magnitude of response that occurs per unit of driving time. This measure cumulates all the galvanic skin responses independently of events and equates the routes on the basis of time. This ratio is, therefore, a statistic reflecting the behavioral response of the driver per unit time and under the assumptions of this study was considered as a measure of induced tension. From table 1 it may be seen that route No. 1 was 45 percent more tension inducing than the alternate route.

In order to test for differences among drivers as well as to determine whether there was a statistically significant difference between the two routes, the statistical technique of analysis of variance was used. A summary of the analysis is shown in table 2. Statistically, both the routes and the drivers differed significantly at the 0.01 level. It would seem reasonable to conclude, therefore, that the alternate route induced significantly less tension in drivers than did the major arterial route. In addition, the combining of the data of the different drivers appears to be unwarranted. This is especially relevant to the GSR data shown in table 1 where average magnitude of the GSR appeared the same for the two routes. Actually, the average magnitude showed considerable variation among the drivers, and the differences in average magnitude of each driver between the routes varied over a range of nearly two-to-one. Thus, the averages shown are quite misleading and, consequently, the combined GSR data must be interpreted with caution.

Table 2.—Summary of analysis of variance of the magnitude of GSR per minute for the test routes

Sources of variance	Sum of squares	Degrees of freedom	Mean square S^2	Variance ratio F
Route	19.57	1	19.57	[1] 11.51
Drivers	302.05	3	100.68	[1] 59.22
Routes and drivers	22.89	3	7.63	[2] 4.48
Residual variance	16.95	10	1.70	- -
Total variance	361.46	17	- - - -	-

[1] Significant at the .01 level. [2] Significant at the .05 level.

Route No. 1

In addition to the study of the continuous runs made over the two routes, each route was studied independently. On the major arterial route, tabulating the occurrence of the traffic events, there was a total of 7,800 traffic events observed during the 2-week period. Data classified by the eight types of traffic event are presented in table 3. The most frequently occurring event was friction with instream moving vehicles. The remainder individually form small proportions of the total. In the last column of the table, the relative rank of each type of traffic event is shown for the average magnitude of GSR which it generated.

To test the reliability of the average magnitude of response, a rank test was applied using the data for each driver independently. In other words, the response generated by each traffic event, for each driver, was ranked and tested. The order shown in table 4 was found to be statistically reliable at better than the 0.01 level.

Data collected during the different traffic periods were also analyzed by direction. It was found that there was considerable variation among the runs made during peak hours and offpeak hours. Table 4 shows the average

Table 3.—Frequency of occurrence and average magnitude of responses evoked by traffic events on route No. 1

No.	Traffic event	Number of events	Percent of total	Average magnitude	Rank by average magnitude
1	Parking maneuvers	724	9.2	2.47	6
2	Marginal pedestrians	330	4.2	1.51	8
3	Instream vehicles	4,682	59.7	2.49	5
4	Loading platforms	633	8.0	2.20	7
5	Instream pedestrians	372	4.7	2.76	4
6	Diverging vehicles	400	5.1	3.34	1
7	Merging and crossing vehicles	285	3.6	3.15	2
8	Traffic signals	416	5.3	2.94	3
	All events	7,842	99.8	2.53	- - -

Table 4.—Effects of traffic period and direction on the average magnitude of response for route No. 1

No.	Traffic event	Northbound			Southbound		
		Offpeak	Peak	Difference	Offpeak	Peak	Difference
1	Parking maneuvers	2.59	2.37	−0.22	2.25	2.87	+0.62
2	Marginal pedestrians	1.10	1.57	+.47	1.88	1.98	+.10
3	Instream vehicles	2.51	2.47	−.04	2.36	2.64	+.38
4	Loading platforms	2.09	1.45	−.64	2.27	3.15	+.88
5	Instream pedestrians	2.63	3.32	+.69	2.29	2.80	+.51
6	Diverging vehicles	3.64	2.86	−.78	3.37	3.49	+.12
7	Merging and crossing vehicles	2.97	3.13	+.16	2.50	3.96	+1.46
8	Traffic signals	3.11	3.67	+.56	2.25	2.65	+.40
	All events	2.50	2.52	+.02	2.36	2.77	+.41

magnitude of response and the changes from offpeak to peak period that occurred in each direction. The plus sign indicates that the response increased during the peak hours, whereas the minus sign shows that it decreased.

For the northbound direction there was almost no difference in average magnitude of response between offpeak and peak runs. In general, the data obtained in the northbound direction were more stable than the data obtained from the southbound runs. Consequently, the differences between peak and offpeak periods are most clearly seen for the former. The greatest increase in response during the peak hours appeared to be in the average tension induced by instream pedestrians (event No. 5). This was consistent with the increase in mass transit use during the peak hours and, consequently, the high density of pedestrians at loading platforms during those hours. Major decreases occurred in the turning movements out of the traffic stream (event No. 6). This, in part, reflects a reduction in the allowable turning movements during peak hours.

The average tension response for all events recorded in the southbound runs increased appreciably from the offpeak to the peak period. The greatest increase occurred in connection with merging and crossing vehicles (event No. 7). Because of the wide variability in the data, only qualitative interpretations are possible.

The analysis of test runs made during night traffic (data not shown here) appeared to follow the pattern derived from the analysis made during the day peak hours. The most obvious differences were in the northbound direction. The only major change at night appeared to be the large increase in response to marginal pedestrians (event No. 2). This was perhaps due to the large increase in pedestrians during evening shopping hours and also the restriction in marginal visibility over much of the street.

Analyzed by control sections

Route No. 1 was divided for analytical purposes into nine control sections of approximately equal length. The 10 control points (for these nine sections) were marked on the recorder tape so that it was possible to relate the events to their location on the route. Table 5 shows the average magnitude of GSR tension response for each section both by direction and traffic period. In addition, figures 2 and 3 show the mean magnitude of tension response by control sections. Generally, there was considerable variability in the data within a section, while the differences between sections were, on the average, not too great. There was no significant order of tension responses found among the sections for the five test drivers. Thus, there was no indication that there were fixed features in any of the sections which consistently influenced driving behavior.

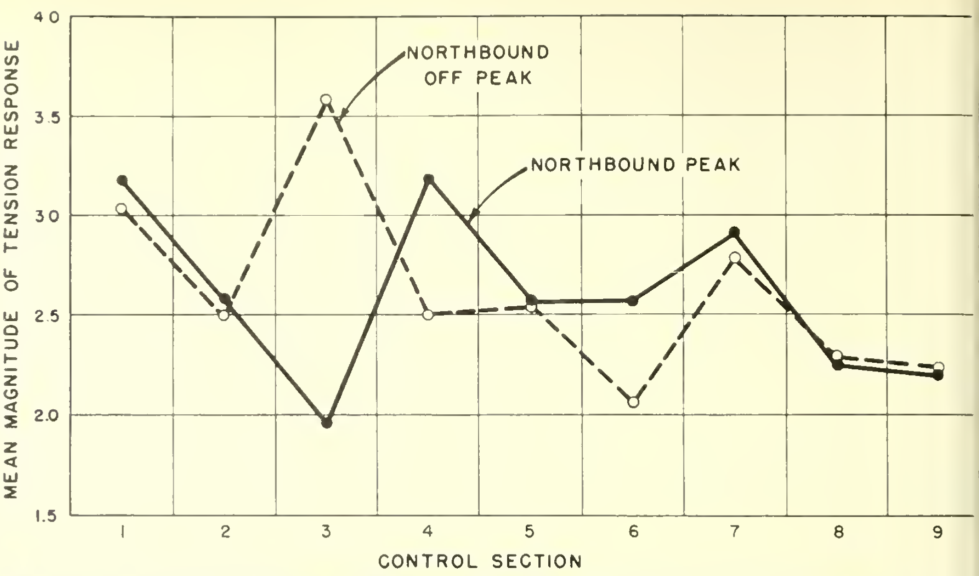

Figure 2.—Average magnitude of tension responses for each control section while travelin in the northbound direction.

As may be seen from figure 2, the differences between peak and offpeak were quite small for all control sections except in section 3. During the offpeak hours this section gave the maximum average tension response, whereas during the peak hours it yielded the lowest average tension response. In order to test for the significance of this difference, a t test was made of each driver's tension responses in peak and offpeak traffic (a t test of matched pairs). Since it was found that the decrease from offpeak to peak was significant at better than the 0.05 level, a real change in traffic characteristics apparently occurred in control section 3. This was an area of intense commercial development and also an area where the street was relatively narrow. During the offpeak hours there was a high degree of marginal activity related to the commercial development. Also, reduced maneuverability due to the narrowness of the street posed a severe restriction on the driver's freedom to make lateral avoidance movements. During peak hours, however, the extensive margin friction was eliminated. Parking regulation high traffic volume, and signal progressi helped to minimize turbulence in the traffi flow.

Table 5.—Average magnitude of responses in the control sections of route No. 1, by direction and traffic period

Control section	Northbound			Southbound			Difference between northbound and southbound	
	Offpeak	Peak	Difference	Offpeak	Peak	Difference	Offpeak	Peak
1	3.04	3.18	−0.14	2.34	2.34	0.00	+0.70	+0.84
2	2.50	2.58	−.08	2.36	2.99	−.63	+.14	−.41
3	3.58	1.95	+1.63	2.47	2.86	−.39	+1.11	−1.91
4	2.50	3.18	−.68	2.18	2.78	−.60	+.32	+.40
5	2.56	2.55	+.01	2.37	3.15	−.78	+.19	−.60
6	2.05	2.57	−.52	2.71	2.89	−.18	−.66	−.32
7	2.78	2.90	−.12	2.82	3.07	−.25	−.04	−.17
8	2.28	2.25	+.03	2.66	2.76	−.10	−.38	−.51
9	2.23	2.20	+.03	2.48	3.06	−.58	−.25	−.86

In the southbound direction there were r clear-cut differences between either traffi period or control section, as may be seen fro figure 3. In control section 7 there did appe to be a maximum average tension respon during the offpeak hours. This section wa located in a highly developed commercial are and there was considerable marginal activit complicated by turning movements into ar from radials at a traffic circle. The pea hours, aside from showing a general rise tension responses, did not indicate any maj differences among the control sections.

Frequency of tension responses

The final analysis of the route No. 1 dat involved the determination of a relative fr quency of traffic events and the total GSR p minute. The pertinent data are shown table 6. These data, which included all th runs for each of the drivers, indicated th there were fewer events per minute in north bound traffic than in southbound traffic. I northbound traffic, the driver encountered event every 25.0 seconds, while traveling the southbound direction the driver e countered a traffic event every 21.3 second Tension inducing events were encounter every 29.4 seconds northbound and 25 seconds southbound. In terms of the measu of induced tension, the magnitude of GSR p unit time, the differences between the tw directions were on the whole quite small a there appeared little evidence to indicate si nificant differences in the two directions. Th drivers also reported that they found n differences between directions.

Route No. 2

In the study of the alternate route (rou No. 2), a different group of driver-subjec were employed, as well as one new observ This route had great variation in grade a

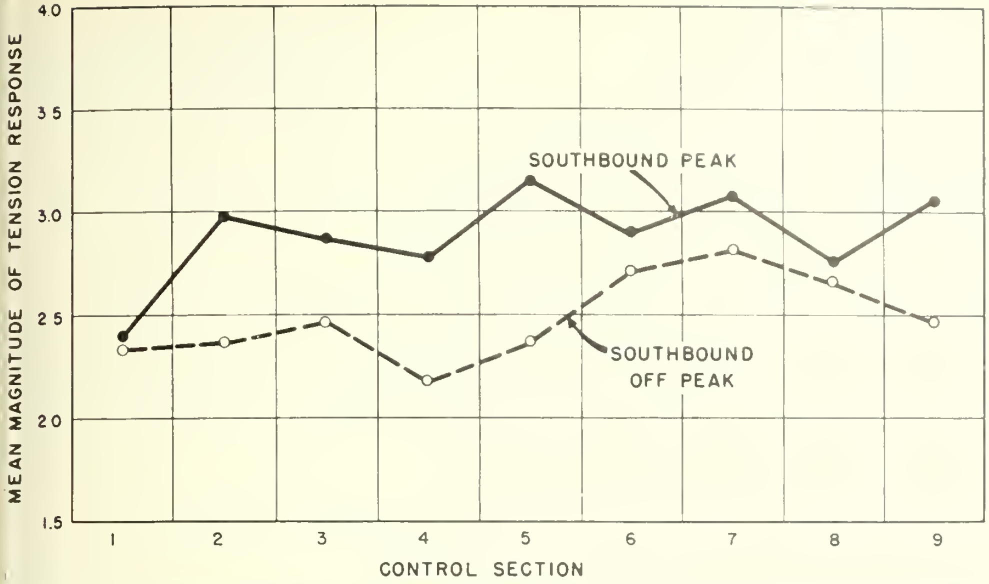

Figure 3.—Average magnitude of tension responses for each control section while traveling in the southbound direction.

width over the 4½-mile length. Unlike route No. 1, there were no streetcars but bus service was in operation in sections of the route. Also in direct contrast to route No. 1, there were no commercial areas. The route was almost wholly through a residential area, consisting for the most part of single-unit dwellings. Again in contrast to route No. 1, where commercial vehicles made up a considerable portion of the traffic, almost all the motor vehicles traveling route No. 2 were passenger cars.

The study method followed for route No. 2 was the same as that used on route No. 1. One change was made in the list of traffic events—the elimination of transit loading platforms and the insertion of opposing vehicles (event No. 4). This event was added because the route was two or three lanes over much of its length. Consequently, there were many possibilities for the opposing traffic stream to come into conflict with the test vehicle.

The number of traffic events and the frequency of occurrence is given in table 7. More than 1,300 traffic events were observed during the study period, and as occurred on route No. 1, the most frequently occurring event was that induced by other vehicles in the traffic stream (event No. 3). The remainder of events individually accounted for only small proportions of the total. Moving vehicles constituted a 6-percent greater part of the total on the alternate route than on the major arterial route. Making the arbitrary assumption of homogeneity, a *t* test of the differences between the two proportions was made. A *t* of 4.21 was obtained which was significant at the 0.01 level. This would appear to indicate that the tension inducing characteristics of the alternate route seemed to include a lesser proportion of peripheral events than that found on route No. 1. Thus, most traffic conflicts on the alternate route appeared to be somewhat more directly related to instream traffic activity.

The ranking of the events according to the average magnitude of response in table 7 shows that the most tension-inducing traffic event was crossing and merging traffic (event No. 7). Opposing vehicles (event No. 4) was second highest in average magnitude of response. In general, the order of intensity of tension induction for each of the events was similar to that found on route No. 1.

The data are shown for the different traffic periods in table 8. The morning offpeak runs are not included in this tabulation as they were made in conjunction with the continuous test runs made on both routes. The overall frequency of occurrence of traffic events was 1.79 per minute, or approximately one every 33 seconds, with a minimum during the morning peak traffic hours of 1.45 events per minute, or one every 41 seconds. The frequency maximum was during the afternoon offpeak traffic hours when there were 2.65 events per minute, or one every 23 seconds. The maximum tension response per minute occurred during the afternoon offpeak hours and was twice as great as the response per minute for either of the peak hour traffic periods. Tension responses per minute for the night runs fell intermediate between the peak and offpeak traffic periods. It should be pointed out, however, that these differences were obtained by pooling data from drivers who were dissimilar in their tension responses. They should, consequently, be interpreted carefully.

Discussion of the Findings

As was pointed out earlier, the purpose of the study was to explore the possibility of using the galvanic skin response as a means for distinguishing features of traffic and streets. The first goal was to detect differences between two arterial routes serving approximately the same traffic function. The results indicated that the GSR reliably discriminated between the two arterial routes, but the distinctions among different characteristics within each route were not as clear cut. However, the results do indicate the complexity of decisions faced by drivers on an urban street. The variety of conflicts occurring on the street were sufficiently frequent and involved to place drivers under a fairly consistent frequency of stress.

One finding of particular interest was, in terms of the average magnitude of GSR, the significant order of the different traffic events. The events which induced the highest average tension on both routes were the conflicts occurring with vehicles entering or leaving the traffic stream and, on the alternate route, the opposing vehicles. These were events in which the rate of change of location of the conflicting vehicles was at a maximum. These were situations where the driver was required to solve a set of differential equations in order to predict a course of action. With the human's limited accuracy in speed esti-

Table 6.—Traffic events and magnitude of tension responses in time on route No. 1

Driver	Events per minute			Magnitude of response per minute		Ratio: northbound / southbound
	Northbound	Southbound	Difference	Northbound	Southbound	
A	2.4	2.9	+0.5	7.30	7.60	0.96
B	2.1	2.5	+.4	6.59	8.30	.79
C	2.7	3.1	+.4	6.91	6.60	1.05
D	2.6	2.7	+.1	4.11	3.81	1.08
E	2.3	2.7	+.4	6.90	9.40	.73
All drivers	2.4	2.8	+.4	6.29	7.23	.87

Table 7.—Frequency of occurrence and average magnitude of responses evoked by traffic events on route No. 2

No.	Traffic event	Number of events	Percent of all events		Average magnitude	Rank by average magnitude
			Route No. 2	Route No. 1 [1]		
1	Parking maneuvers	70	5.3	9.2	1.30	8
2	Marginal pedestrians	9	0.6	4.2	1.56	7
3	Instream vehicles	863	65.6	59.7	1.77	5
4	Opposing vehicles	50	3.8	[2] 8.0	2.14	2
5	Instream pedestrians	17	1.7	4.7	1.71	6
6	Diverging vehicles	70	5.3	5.1	1.79	4
7	Crossing and merging vehicles	93	7.0	3.6	2.58	1
8	Traffic signals	142	10.8	5.3	1.97	3
	All events	1,314	100.1	99.8	1.84	--------

[1] Data from table 3. [2] Event No. 4 on route No. 1 was transit loading platforms.

mation and angular closing rate, and the limited time for such decision, these situations had a high degree of unpredictability for the driver and may reasonably be most threatening.

The two events which ranked next in order were the traffic signals and instream pedestrians. Both may be considered as "instream uncertainty." In the study, the driver was influenced by traffic signals only when there were no other vehicles interposed between him and the signal. There was, then, a fairly high probability that the driver arrived at a signal at the moment when it had just changed or was in the process of changing. This would appear to be a particularly indeterminate situation for the driver. The effect of instream pedestrians was quite similar. The driver had no way to predict the action of a pedestrian. Any conflict would arise strictly by action of the pedestrian. Both events probably represent a straight risk-type decision operation, and the observed magnitude of tension response may well reflect the degree of uncertainty in each situation.

A third pair of traffic events which appear to go together are the instream vehicle and parking maneuver events. These both may be considered "instream interferences." In the instream vehicle event, the vehicles were moving in the same direction and the relative differences in velocity between the test vehicle and other vehicles in the stream was relatively small. Consequently there was adequate time for compensation for any changes in the characteristics of the ongoing traffic stream. Parking maneuvers were generally found to be quite conspicuous so that the test driver was able to adapt his speed or direction with relative ease. Both events appeared to be relatively predictable actions for which the driver could adequately compensate, and for which there was adequate time for decision making.

The last two traffic events, marginal pedestrians and the streetcar loading platforms, may be termed "fixed objects." For all cases observed, conflict between pedestrian and vehicle occurred when the pedestrian was standing or just beginning to move into the street. At this point it seems reasonable to consider this as a fixed-obstacle situation. In general, it appeared in both of these types of events that the driver had more or less complete control over his actions. Thus, these events may be conceived as simply choice points, and the responses were a reflection of tension stress due to a choice made by the driver.

Predictability and response

One general implication from the results of the study is consistent with current knowledge of the GSR. It is that the more highly unpredictable the situation, the more dynamically the subject responds. The unique thing in the study was the fact that there appeared to be a very high level of unpredictability in driving on urban streets. The data indicated that about three times a minute an event occurred which forced the driver to take some compensatory action. Furthermore, as the traffic situation created a more complex demand upon the driver, the tension aroused by that situation became even greater.

Table 8.—Frequency of occurrence and average magnitude of response on route No. 2
time period

Traffic period	Number of events	Average magnitude of response	Events per minute	Magnitude of response per minute
Morning peak	292	1.68	1.45	2.44
Afternoon peak	363	1.41	1.86	2.62
Afternoon offpeak	122	1.98	2.65	5.25
Night	346	2.21	1.89	4.18
All traffic	1,123	1.79	1.79	3.20

These results indicate that the driving environment generates a tension response in inverse relationship to the predictability of the conflicts. The ranking order of the traffic events significantly demonstrates the situations which are hardest for the driver to predict and thus to compensate for. Also, the complexity of the driving environment may be added to the element of predictability. In the only control section where there was a significant difference between traffic time periods, the decrease in average GSR came when the complexity of the traffic situation was reduced.

In addition, the differences between the two arterial routes, in terms of their percentage of marginal interferences, also indicated a difference in complexity between the two routes. On route No. 1 over 40 percent of all observed events arose from interferences occurring along the margins of the street. In such a traffic situation the driver is forced to attend to a wide range of stimuli. He must sort, select, and then operate on this heavy load of information. His ability to select and predict is inherently restricted; one consequence of which is an increased level of stress and a greater tension response to events.

Delegation of control to other drivers

Finally, analyses of the findings indicated a high rate of decision making for drivers on an urban street. Where the driver was forced to respond to a fixed object the average GSR was relatively low. This was a situation where the driver usually made the decision on his own terms in his own time. In the situation of vehicles entering or leaving the traffic stream, the driver was forced to make decisions both very rapidly and with a minimum amount of information, much of which he could not handle accurately or efficiently. Under these circumstances the driver was dependent upon other drivers to respond consistently. In complex traffic, then, the individual driver was often forced to give up a certain amount of control to other drivers in the traffic stream.

As to how this delegation of control would affect traffic operations and capacity, it appears reasonable to believe that a driver will compensate for this loss of control by any means at his disposal that will reduce his uncertainty. He can, for example, increase the headway between himself and the vehicle ahead, or by reducing his speed. Such compensations, however, have a tendency to reduce the capacity of the street, and cause turbulence in the flow of traffic.

Reliability of the GSR

The results of the study indicated that t GSR was an adequate measure of driver b havior. There were, however, questions rela ing to the GSR which have not been discusse in this article. One thing that needs to be e amined far more intensively is the reliabili of the galvanic skin response (*6*). It is we known, for example, that there is a rath consistent adaptation of the GSR so that t same stimulus intensity may not arouse t same magnitude of GSR on repetition (*7*). T was in part compensated by the calibratio procedure used in the study, and the fact th no traffic event occurred twice in precisely t same way. Nevertheless, the changes whi occurred during a test run were not examin in detail.

Some very simple assumptions have be made about the relation of the tension r sponses to the traffic situations. The proble is essentially one of determining what t responses mean in a traffic situation. It is a oversimplification to assume, as was done this study, that the GSR is aroused only b the occurrence of a traffic event. The GSR sensitive to a wide variety of behavior responses (*8*) whose relationships may be on indirectly related to the traffic event. The is little doubt, for example, that the GSR a companies preparatory muscular activity muscular response itself (*9*).

Statistical significance of a tension response

Another problem is the statistical nature the galvanic skin reflex. In the study t magnitude of response was a positively skewe distribution. The range of conductance w from zero to some maximum value. Such di tributions pose some difficult problems statistical analysis. Thus, in this study, t use of the arithmetic mean and the usu statistical tests of inference are in question.

There are also certain methodological prob lems of specifying and interpreting the traffi events. For the purposes of this study eac traffic event was treated as discrete or isolate It is obvious, however, that conflicts in traffi are not discrete but develop continuously time. Thus, the schedule of observation arb trarily collapsed a complex and continuou behavioral response to a single point in tim There is no way of knowing from the presen study whether an observed galvanic skin r sponse occurred at the approach to a conflic during the conflict itself, or at the point time when a decision was made. It is con ceivable that any or all of these process could evoke a response.

(*Continued on page 71*)

Traffic Operations as Related to Highway Illumination and Delineation

A Cooperative Investigation by the Connecticut State Highway Department and the Bureau of Public Roads

Reported [1] by
ASRIEL TARAGIN, Chief, Traffic Performance Branch, Division of Traffic Operations, Bureau of Public Roads, and
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The increasing mileage of freeways being put into operation has stimulated much discussion of highway illumination and its possible value in reducing the tension and strain of night driving and in reducing traffic accidents, and perhaps thereby increasing the night usage of these highways. Because of lack of factual knowledge on the subject, the Connecticut State Highway Department in cooperation with the Bureau of Public Roads undertook a study of the effects of illumination and delineation on the Connecticut Turnpike. Driver behavior data were recorded under nine different conditions of highway illumination and delineation at a heavily traveled interchange illuminated with mercury lamps. Accident data were obtained on the 53-mile continously illuminated section and the 77-mile nonilluminated section of the Turnpike.

The study showed no significant differences with respect to average vehicle speeds, lateral placements, and clearances between vehicles, under the various conditions of illumination and delineation. The manner of night usage of speed change lanes, particularly the acceleration lane, improved with increased illumination. In general, it appeared that some benefit resulted from full-level illumination in the deceleration area, and that even greater benefit occurred when illumination was combined with roadside delineation. Illumination of the interchange area only did not appear to be advantageous insofar as the access ramp site was concerned. The importance of delineation, with or without illumination was demonstrated.

TRAFFIC at night has always had accident rates averaging about twice that of dayime rates, and the rapid, continuing growth of travel has increased both the awareness of he magnitude of this problem and the efforts o develop remedial measures. Night driving nvolves not only the problems of darkness, but hazards of fatigue, drowsiness, and other factors. The ultimate solution in avoiding darkness, of course, would be to illuminate the roadways to the same intensity as exists during daylight, but this is obviously impractical. It remains to be determined, therefore, what level of illumination would induce drivers to operate their vehicles at night in the same manner as they do during daytime.

While it has been an accepted fact that good visibility is a prerequisite to good traffic operations, there have been no accredited warrants set forth for highway lighting. Similarly, there has been no correlation beween the effects on traffic operations of deineation (reflector buttons) and illumination.

Scope of Study

The importance of highway lighting was emphasized at the 1958 annual meeting of the Highway Research Board and again at the 1959 annual meeting of the American Association of State Highway Officials. Because very little was known on the effectiveness of highway lighting on freeways, with respect to driver behavior, accidents, and night usage of the highway, the Connecticut State Highway Department in cooperation with the Bureau of Public Roads undertook a comprehensive study in this field in 1959. Continuous modern highway lighting on a 53-mile section of the 129-mile Connecticut Turnpike afforded a good opportunity to investigate the effects of illumination and delineation on traffic operations.

The purpose of the study was to evaluate the effectiveness of roadside delineation, pavement markings, and a combination of delineations and markings under conditions of full, partial, and no highway lighting. The effects to be ascertained would be those manifested in accidents, and in drivers' actions such as speed, lateral placement, headway distance, lane usage, and utilization of acceleration and deceleration lanes. A total of 183,000 motor vehicles were observed under the following nine principal lighting conditions:

No illumination:

1. Lane lines only.
2. Lane lines and edge lines.
3. Lane lines, edge lines, and roadside delineators.

Partial illumination:

4. Lane lines, edge lines, and one-half normal illumination.
5. Lane lines, edge lines, delineators, and one-half normal illumination.
6. Lane lines, edge lines, delineators, and one-half normal illumination in interchange area only.

Full illumination:

7. Lane lines, edge lines, delineators, and normal illumination in interchange area only.
8. Lane lines, edge lines, and normal illumination.
9. Lane lines, edge lines, delineators, and normal illumination.

The white reflectorized lane lines were dashed, with both dashes and spaces being 25 feet in length. All edge lines were reflectorized; solid white to the left of the traffic stream and solid yellow to the right. The lane and edge lines were 6 inches in width on the Turnpike proper, and 4 inches in width on the access and exit ramps.

Highway lighting consisted of mercury luminaires throughout. Delineation consisted of acrylic plastic reflex reflectors, 3 inches in diameter, mounted at a height of 4½ feet above the pavement. The reflectors were spaced at 200-foot intervals on the Turnpike and at reduced intervals on the access and exit ramps. They were placed about 12 feet from the pavement edge on the shoulder and about 5 feet from the pavement on the median strip. Installations on the Turnpike consisted of single white reflectors on both sides of the roadways, whereas dual amber reflectors were used on both sides of the ramps.

Locations Studied

Although originally planned procedures included a number of study sites, preliminary work indicated the necessity for simplification of the study program if it was to be completed within a reasonable time period. Two comparable sites were selected for detailed study, located at a Turnpike interchange with large volumes of traffic. Site 4M was located on the access ramp and site 5M on the exit ramp. From all the sites originally selected for study, it was presumed that site 4M and

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[1] This article was presented at the 39th Annual Meeting of the Highway Research Board, Washington, D.C., January 1960.

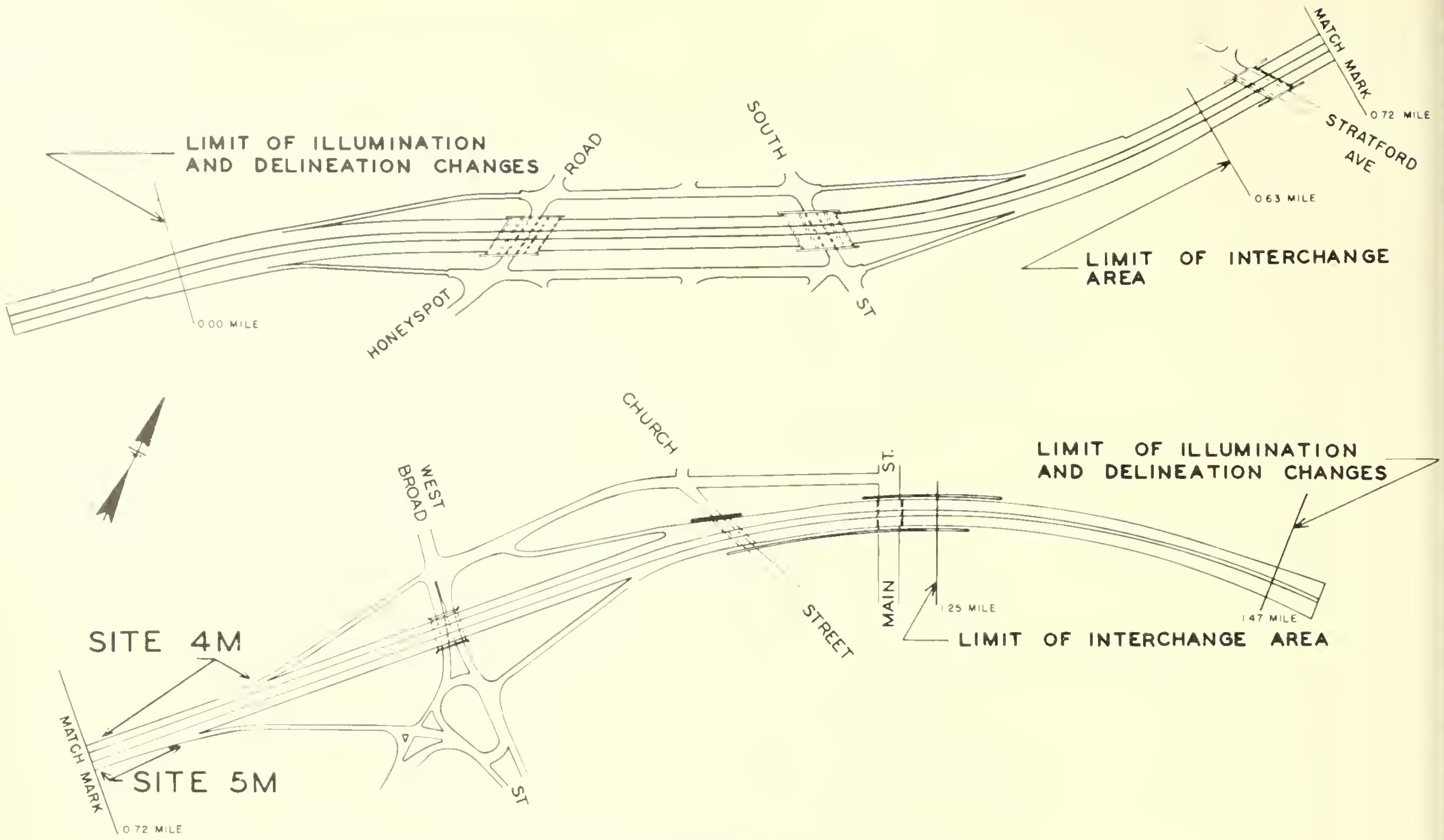

Figure 1.—Plan of general study area.

5M would reveal the most significant effects evidenced by the nine variable conditions of illumination and delineation.

From the New York State line to 8 miles east of New Haven, a distance of 53 miles, the Connecticut Turnpike was illuminated to a level of 0.8 foot-candle on the roadway and access ramps. The exit ramps were illuminated at a somewhat lower level. The roadway consisted of three 12-foot concrete lanes on either side of a median that varied in width from 4 feet to 30 feet.

At the study location, the West Broad Street interchange in the town of Stratford, 15 miles southwest of New Haven, the six lanes were abutted by 2-foot-wide bituminous gutter strips and 10-foot-wide bituminous shoulders. The depressed grass median was 30 feet in width and the access ramp and exit ramp roadways were 13 feet wide. Average daily traffic volumes on the Turnpike just west of the study interchange were 17,000 vehicles in each direction. At the interchange, almost 7,000 vehicles left the Turnpike and over 7,000 vehicles entered daily.

The general area of the interchange, in which the nine lighting conditions were established for the study, is shown in figure 1. Also shown are the limits of illumination and

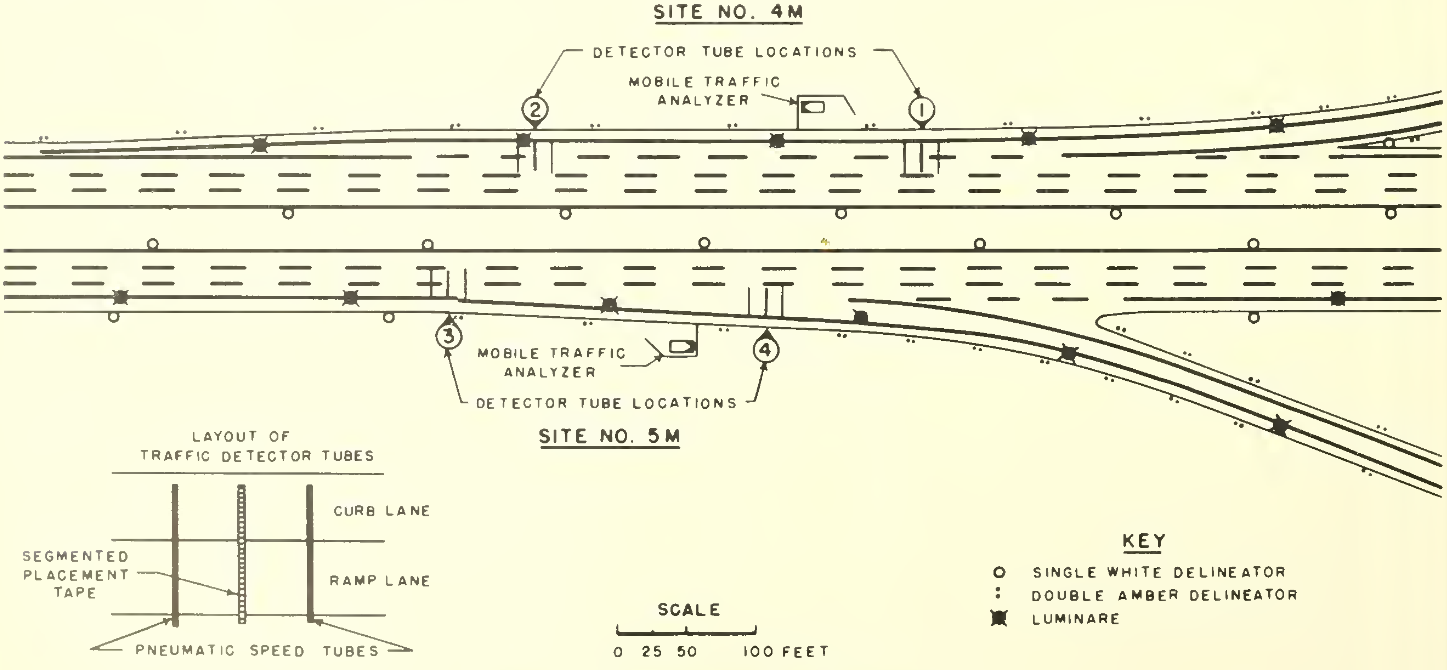

Figure 2.—Plan of specific study locations.

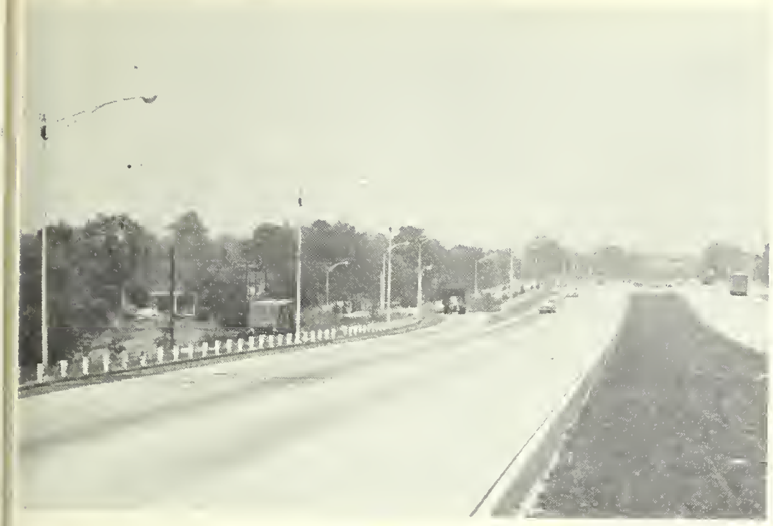

Figure 3.—Access ramp site 4M.

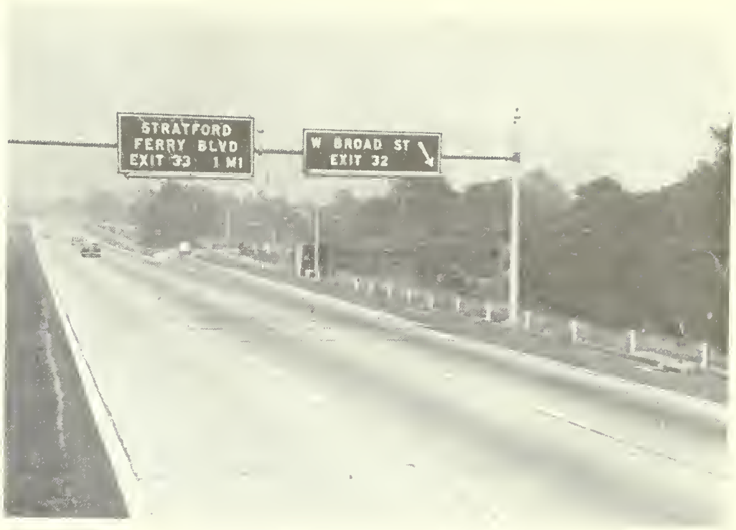

Figure 4.—Exit ramp site 5M.

delineation changes for the eastbound and westbound traffic, and the limits of the interchange area. Lights were controlled within these limits.

Specific observations were made at both the westbound access ramp, site 4M, and the eastbound exit ramp, site 5M. Figure 2 shows the roadway, illumination, delineation, and locations of the recording and detecting equipment in the vicinity of the study sites. Figure 3 shows the access ramp site (4M) looking easterly along the westbound lanes. Figure 4 shows the exit ramp site (5M) looking westerly along on the eastbound lanes.

Partial illumination (for lighting conditions 4, 5, and 6) was attained by changing lamps and ballasts as well as by using specially constructed lamps with the existing ballasts. The actual levels of illumination achieved were determined by the standard practice for measurement as recommended by the Illuminating Engineering Society. The actual values were measured at intervals of 10 feet by the light meter for the study area. For illustrative purposes, however, only typical average foot-candles of illumination are shown in figure 5 for normal illumination and in figure 6 for partial illumination. Generally, the illumination ratio of the average to the minimum was 4 or 6 to 1. The different appearances of the highway under the 2 levels of illumination were readily discernable visually as shown in figure 7.

Field Observations

The mobile traffic analyzer designed and constructed by the Bureau of Public Roads was used in the study to record the traffic data. Constructed from a revamped delivery truck, the traffic analyzer vehicle housed four solenoid-operated adding machines, a digital clock, telegraph keys, and other supporting equipment, shown in figure 8. All of the equipment was electronically interconnected.

The recording units in the vehicle were connected by multiconductor cables to four sets of road-detector tubes placed across the roadway at four specific locations, shown in figure 2. One set of detector tubes was placed across the curb lane and access ramp lane near the beginning (location No. 1) and a second set near the end (location No. 2) of the acceleration lane (site 4M); the third set of detector tubes was placed across the curb and middle lanes near the beginning of the deceleration lane (location No. 3); and the fourth set was placed across the curb lane and exit ramp lane (location No. 4) near the gore area (site 5M). As the motor vehicles on the Turnpike passed over the detector tubes, electronic impulses were transmitted to the traffic analyzer and recorded.

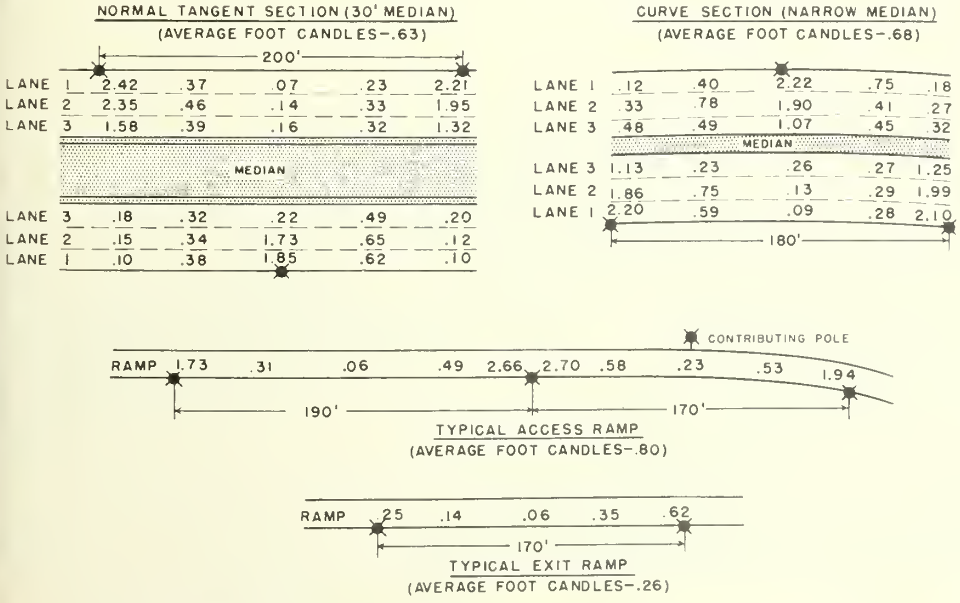
NORMAL TANGENT SECTION (30' MEDIAN)
(AVERAGE FOOT CANDLES—.63)
200'
LANE 1 2.42 .37 .07 .23 2.21
LANE 2 2.35 .46 .14 .33 1.95
LANE 3 1.58 .39 .16 .32 1.32
MEDIAN
LANE 3 .18 .32 .22 .49 .20
LANE 2 .15 .34 1.73 .65 .12
LANE 1 .10 .38 1.85 .62 .10
CURVE SECTION (NARROW MEDIAN)
(AVERAGE FOOT CANDLES—.68)
LANE 1 .12 .40 2.22 .75 .18
LANE 2 .33 .78 1.90 .41 .27
LANE 3 .48 .49 1.07 .45 .32
MEDIAN
LANE 3 1.13 .23 .26 .27 1.25
LANE 2 1.86 .75 .13 .29 1.99
LANE 1 2.20 .59 .09 .28 2.10
180'
CONTRIBUTING POLE
RAMP 1.73 .31 .06 .49 2.66 2.70 .58 .23 .53 1.94
190'
170'
TYPICAL ACCESS RAMP
(AVERAGE FOOT CANDLES—.80)
RAMP .25 .14 .06 .35 .62
170'
TYPICAL EXIT RAMP
(AVERAGE FOOT CANDLES—.26)

Figure 5.—Measured illumination values under normal illumination.

The data electronically recorded included the speed of the motor vehicle, lateral placement (from which lateral clearances between vehicles were calculated), and time of day to the nearest one ten-thousandth of an hour (from which traffic maneuvers and headway distances were calculated).

A supervisor and four observers were located inside the traffic analyzer vehicle and four more observers were stationed outside, but in the immediate vicinity of the vehicle. The inside observers operated certain keys which recorded vehicle classification and driver action in the merging and diverging traffic areas. The outside observers recorded the manner in which drivers utilized the acceleration and deceleration lanes including various driving actions such as use of brakes, sudden slowdowns, etc. They also recorded traffic volumes and lane changing on those lanes not equipped with road-detector tubes.

Accident data, compiled for both illuminated and nonilluminated areas, were obtained from State Police accident reports. The only Turnpike accidents considered for comparative analyses were accidents which occurred during the first 8 months of 1959. Turnpike accidents prior to 1959 were not considered indicative of the accident experience because the road was only partially open to traffic in 1958. Accident information for the Merritt Parkway and for all other Connecticut State highways, used for comparative purposes, was obtained from the last full year of accident tabulations available from motor vehicle reports.

Analyses and Discussion of Data

The multitude of data recorded could not be treated by normal tabulating procedures and a high-speed electronic computer was employed. For simplicity in analyses of ve-

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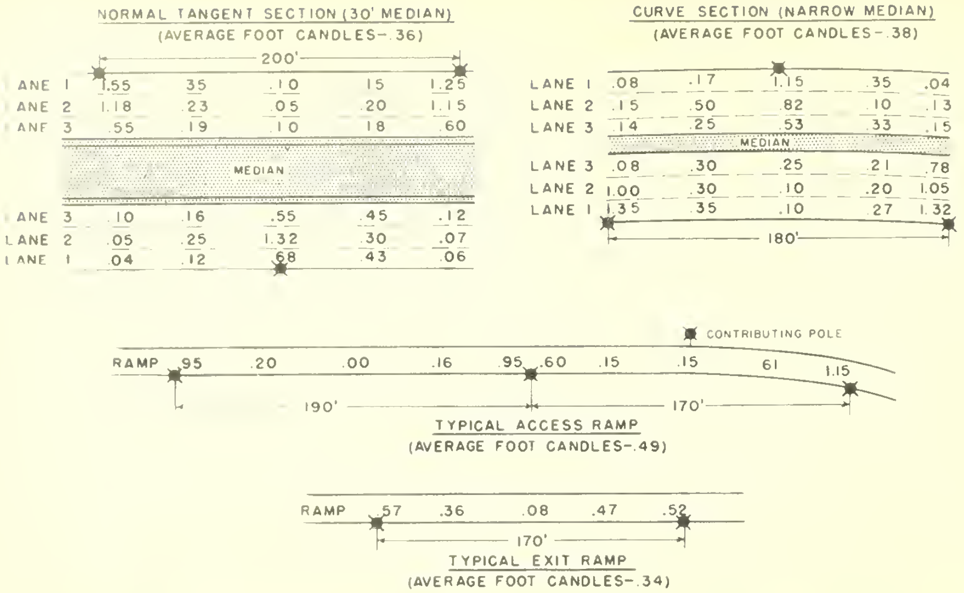

NORMAL TANGENT SECTION (30' MEDIAN)
(AVERAGE FOOT CANDLES-.36)

CURVE SECTION (NARROW MEDIAN)
(AVERAGE FOOT CANDLES-.38)

TYPICAL ACCESS RAMP
(AVERAGE FOOT CANDLES-.49)

TYPICAL EXIT RAMP
(AVERAGE FOOT CANDLES-.34)

200'
ANE 1 1.55 35 .10 15 1.25
ANE 2 1.18 .23 .05 .20 1.15
ANE 3 .55 .19 .10 18 .60
MEDIAN
ANE 3 .10 .16 .55 .45 .12
LANE 2 .05 .25 1.32 .30 .07
LANE 1 .04 .12 68 .43 .06
LANE 1 .08 .17 1.15 .35 .04
LANE 2 .15 .50 .82 .10 .13
LANE 3 .14 .25 .53 .33 .15
MEDIAN
LANE 3 .08 .30 .25 .21 .78
LANE 2 1.00 .30 .10 .20 1.05
LANE 1 1.35 .35 .10 .27 1.32
180'
CONTRIBUTING POLE
RAMP .95 .20 .00 .16 .95 .60 .15 .15 61 1.15
190'
170'
RAMP .57 .36 .08 .47 .52
170'

Figure 6.—Measured illumination values under one-half normal illumination.

hicle speeds and lateral placements, motor vehicles were grouped into two categories: (1) light vehicles—consisting of passenger cars and 2-axle, single-tire trucks; and (2) heavy vehicles—consisting of all heavier trucks, combinations, and buses. The vehicles were also classified according to their proximity to other vehicles on the roadway, as follows:

Free-moving vehicle.—One whose longitudinal spacing to the nearest vehicle, ahead or behind and in any lane, was more than 7 seconds.

Adjacent vehicle.—One whose longitudinal spacing to the nearest vehicle in an adjoining lane was 1.4 seconds or less.

Trailing vehicle.—One whose longitudinal spacing to the preceding vehicle in the same lane was 3 seconds or less.

Adjacent and trailing vehicle.—One whose longitudinal spacing was 1.4 seconds or less to the nearest vehicle in an adjoining lane, and at the same time less than 3 seconds behind a preceding vehicle in the same lane.

Other.—One whose longitudinal spacing to another vehicle in the same lane was over 3 but less than 7 seconds; or whose longitudinal spacing to another vehicle in an adjoining lane was more than 1.4 but less than 7 seconds.

The vehicle position classifications thus include free-moving and unaffected; adjacent to a vehicle in the next lane; trailing another vehicle and either adjacent to another vehicle or laterally in the clear; and finally, "other," representing a position of near but not close proximity to other vehicles.

Traffic volumes

The total traffic and the traffic distribution by lanes are shown in table 1. With the exception of one period during nighttime traffic at site 4M (lighting condition No. 8, lane and edge lines and normal illumination), the access and exit ramps carried more traffic than any other lane. Generally the median lane carried the smallest percentage of vehicles and the curb lane the next smallest.

At both sites the percentage of heavy trucks and buses was greater at night than during the day, as shown in table 2. Heavy trucks and buses accounted for 12 percent of the vehicles recorded at site 4M and 10 percent recorded at site 5M. The smaller percentage of heavy vehicles recorded at site 5M was due to the location of a toll plaza east of the study site and the preceding exit ramp west of the study site which led directly to U.S. Route 1 in Stratford. From an analysis of traffic data (not shown in tables) it was found that a higher percentage of heavy vehicles were in the through-traffic streams.

Vehicle speeds

No significant relation between vehicle speeds and the lighting conditions was established by the study. Variations among daytime speeds were as great or greater than between day and night vehicle speeds; and these variations were found for free moving, adjacent, and trailing light and heavy vehicles. A summary of the average day and night speeds recorded at the four locations is given in table 3.

Classified by the nine lighting conditions during day and night traffic, the difference between the average speeds rarely exceeded 3 miles per hour, and generally the difference was less than 1 mile per hour. A representative comparison of the speeds of light vehicles on through traffic lanes, their lateral placements, and the clearances between adjacent vehicles, is shown in table 4. Cars and light trucks observed in the curb lane at site 4M and in the middle lane at site 5M were used for the comparison because these lanes carried the greatest proportion of through vehicles.

A review of the distribution of all speeds (of which table 4 is but a representative sample) indicated that the percentage of through light vehicles traveling below 40 miles per hour could provide an index for comparing the relative advantages of the study lighting conditions. The index derived for each lighting condition was the variation (expressed in percent) of the percentage of nighttime vehicles traveling below 40 miles per hour from the percentage of the total daytime vehicles observed at the same site traveling below 40 miles per hour during the average day. For example, if 35 percent of the cars and light trucks traveled below 40 miles per hour during the average day and 30 percent traveled below 40 miles per hour at night, the percent variation was (30−35)÷35 or 14 percent. If the nighttime percentage of vehicles traveling below 40 miles per hour was lower than the daytime figure, then the variation was minus. If the converse were true, the variation would be plus.

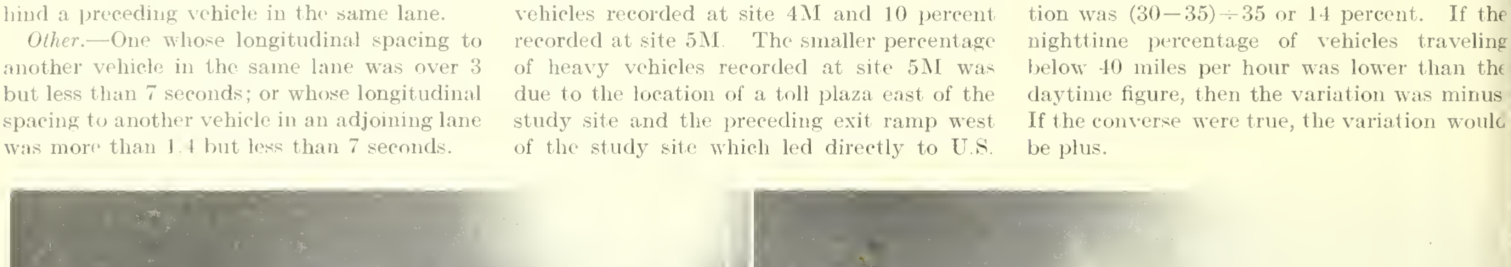


Figure 7.—A section of the Connecticut Turnpike under one-half normal illumination (left) and normal illumination (right).

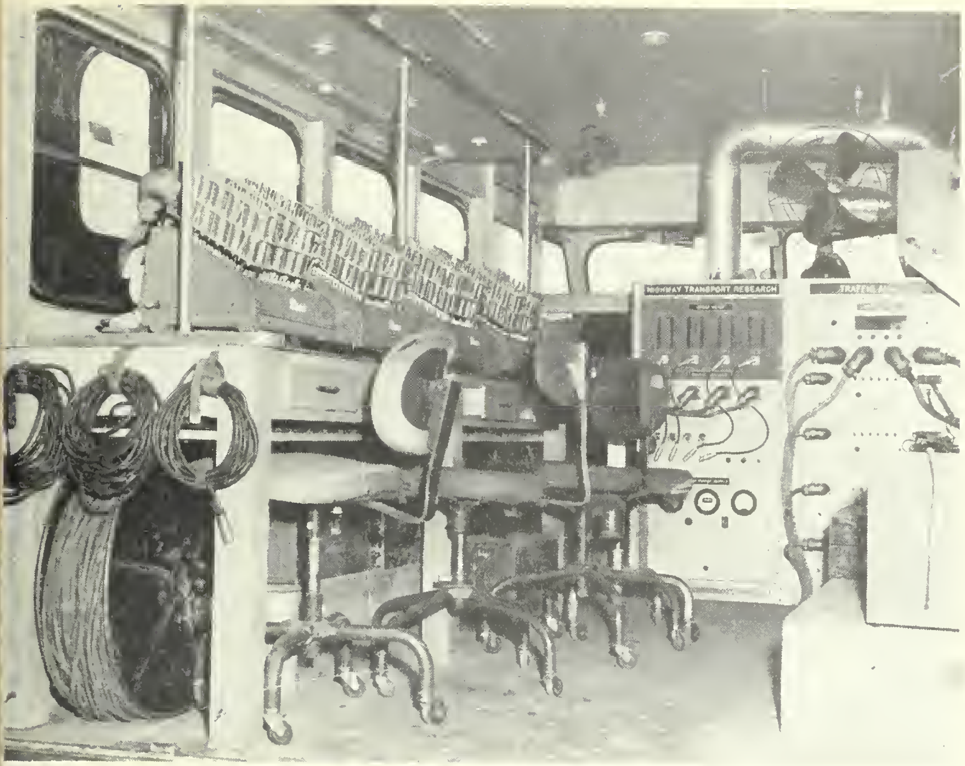


Figure 8.—Interior view of the mobile traffic analyzer vehicle.

In simpler words, a minus variation means at a larger proportion of vehicles were aveling faster at night than in daytime.

The seemingly large values that appear for me lighting conditions, shown in figures 9 id 10, resulted usually from the fact that iring the daytime the average percentage of)served speeds below 40 miles per hour was iite low. For site 4M, the least percentage iriation occurred for the lighting conditions full illumination with and without delineation (condition Nos. 8 and 9), partial illumination with delineation (condition No. 5), and delineation but no illumination (condition No. 3). From the variations it would appear that roadside delineation exhibited an influence toward maintaining minimum speed differentials; and that where delineations were present, partial illumination exhibited very little improvement over no illumination.

The data for site 5M, shown in figure 10, revealed an entirely different trend. (The great difference in vertical scale between figures 9 and 10 should be noted.) A smaller percentage of through light vehicles traveled below 40 miles per hour during night traffic than during the day for all lighting conditions except condition No. 4, edge and lane lines and one-half normal illumination. One explanation given for the exception was that the reduced illumination in the area of heavy diverging maneuvers resulted in a loss of confidence and subsequent lowering of speed.

To test the results given in figures 9 and 10, cumulative speed curves for each lighting condition were compared to the similar curve for the average day at the two study sites. The findings of the comparison lent support to the effects of lighting determined by speed variations.

Lateral placement

Analyses of the data recorded for lateral placement of vehicles failed to reveal any significant differences for the nine lighting conditions. Certain trends were indicated by the analyses of average day versus night placements, as shown in figures 11 and 12, but none that could be used as criteria for evaluation of the lighting conditions.

In this study, the lateral placement of a vehicle was the distance from the center of the vehicle to the right edge of the traffic lane. In figure 11, for light vehicles, the average placement, averaged for all lighting conditions, showed that at the start of the exit ramp and near the end of the access ramp there was considerable difference between the placements in the outer lane and the inner lane for both day and night traffic. This difference, however, did not appear to exist in the average placement of heavy vehicles (fig. 12).

Analysis of the placement data did indicate that all vehicles in the ramp lanes at both sites

Table 1.—Distribution of traffic by lanes under different highway lighting conditions

Highway lighting condition number and code [1]	Daytime traffic					Nighttime traffic				
	Vehicles per hour	Percent of vehicles using—				Vehicles per hour	Percent of vehicles using—			
		Ramp lane	Curb lane	Middle lane	Median lane		Ramp lane	Curb lane	Middle lane	Median lane
Site 4M:										
No illumination:										
1.—LL [2]	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------
2.—L-EL	672	51.0	22.6	21.3	5.1	313	48.4	29.5	18.4	3.7
3.—L-EL,D	1,236	36.8	17.8	34.0	11.4	491	35.6	27.7	29.8	6.9
Partial illumination:										
4.—L-EL,½I	1,219	42.4	19.1	27.6	10.9	519	35.2	23.1	27.0	14.7
5.—L-EL,D,½I	1,174	38.7	17.1	34.7	9.5	580	40.0	20.8	31.6	7.6
6.—L-EL,D,½I(int.)	1,264	40.6	19.2	30.8	9.4	553	37.0	21.7	34.4	6.9
Full illumination:										
7.—L-EL,D,I(int.)	1,096	40.5	20.0	30.3	9.2	533	38.5	23.1	30.4	8.0
8.—L-EL,I	798	52.0	21.6	21.2	5.2	376	28.5	47.0	20.0	4.5
9.—L-EL,D,I	1,296	39.8	19.0	31.1	10.1	576	40.3	22.0	30.9	6.8
Average	1,094	42.7	19.6	28.9	8.8	494	37.9	26.9	27.8	7.4
Site 5M:										
No illumination:										
1.—LL [2]	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------
2.—L-EL	629	38.8	17.2	32.6	11.4	420	48.9	21.4	24.5	5.2
3.—L-EL,D	1,503	47.2	10.7	26.0	16.1	666	44.2	20.8	27.9	7.1
Partial illumination:										
4.—L-EL,½I	1,509	51.2	10.6	22.8	15.4	570	45.1	19.8	24.5	10.6
5.—L-EL,D,½I	1,488	48.6	10.7	24.5	16.2	656	49.2	14.8	26.9	9.1
6.—L-EL,D,½I(int.)	1,478	48.4	11.7	25.4	14.5	563	47.0	22.0	24.4	6.6
Full illumination:										
7.—L-EL,D,I(int.)	1,567	48.4	10.8	25.5	15.3	582	43.8	22.0	27.2	7.0
8.—L-EL,I	1,401	46.5	10.7	27.1	15.7	810	59.8	18.4	17.5	4.3
9.—L-EL,D,I	1,443	47.7	10.8	25.6	15.9	588	48.5	17.8	27.2	6.5
Average	1,377	47.1	11.6	26.2	15.1	607	48.3	19.6	25.0	7.1

[1] Code: LL—lane lines only; L-EL—lane and edge lines; D—roadside delineation; ½I—one-half normal highway illumination; (int.)—interchange area only; I—normal highway illumination. [2] Traffic data were not available.

Table 2.—Percent of heavy vehicles recorded at the study sites, classified by lighting condition

Highway lighting condition number and code [1]	Percent of total traffic volumes			
	Site 4M		Site 5M	
	Day	Night	Day	Night
No illumination:				
1.—LL	22	23	12	17
2.—L-EL	16	17	10	13
3.—L-EL, D	10	15	7	12
Partial illumination:				
4.—L-EL, ½I	11	15	7	13
5.—L-EL, D, ½I	16	13	9	10
6.—L-EL, D, ½I (int.)	11	11	9	13
Full illumination:				
7.—L-EL, D, I (int.)	11	11	5	8
8.—L-EL, I	14	12	10	15
9.—L-EL, D, I	9	12	8	8
Average	12	13	9	12

[1] For lighting code, see table 1, footnote 1.

Table 3.—Summary of average day and night speeds under all lighting conditions recorded at the four locations

Location	Light vehicles		Heavy vehicles	
	Daytime speed	Nighttime speed	Daytime speed	Nighttime speed
Site 4M:				
Location No. 1:	*m.p.h.*	*m.p.h.*	*m.p.h.*	*m.p.h.*
Ramp lane	38.9	38.8	31.4	29.1
Curb lane	50.0	49.2	48.4	47.7
Location No. 2:				
Ramp lane	41.0	40.3	34.0	30.8
Curb lane	47.7	46.2	47.3	46.4
Site 5M:				
Location No. 3:				
Curb lane	42.3	42.8	45.5	47.3
Middle lane	57.6	57.4	55.9	56.4
Location No. 4:				
Ramp lane	37.1	38.1	37.1	39.4
Curb lane	48.5	49.4	49.5	53.1

tended to travel closer to the through lanes at night; and that all vehicles in the curb lane at both sites generally traveled closer to the right edge of the traffic lane at night.

Headway distance

It was believed that the position a driver selected for his vehicle, in relation to the preceding vehicle in the same traffic lane, would be influenced by various lighting conditions and thereby would afford the researchers an opportunity to measure driver behavior under each lighting condition. The approximate time equivalent of the recommended safe distance between successive vehicles at the posted speed of 60 miles per hour is 1.4 seconds. The percentage of headways below this longitudinal time spacing was therefore selected as the criterion in evaluating the effect of lighting conditions. However, a review of the percentage distributions could not establish any definite relation between lighting effects and headway.

Clearances between vehicles

As utilized in this study, clearance relates to the lateral distance in feet between bodies of adjacent vehicles. The clearances of through light vehicles, shown in table 4, were generally representative of all vehicles under all lighting conditions. Clearance appeared to be greater at site 5M than at site 4M under all but one of the lighting conditions. T
differences in measurement recorded at ea
site, however, were quite small in magnitu
and so varied among the lighting conditio
that no distribution significance could
realized.

Lane usage

The most significant findings of the stu
related to the use of the acceleration a
deceleration lanes. Data for this phase
the analyses were obtained by the observe
stationed outside the mobile traffic-analyz
vehicle. These observers recorded the numb
of vehicles utilizing the access ramp and t
proportion of the length of the accelerati
lane which was traversed by each vehi
before crossing into the curb lane. At t
exit site, vehicles using the deceleration la
and the length of use, were also recorded.

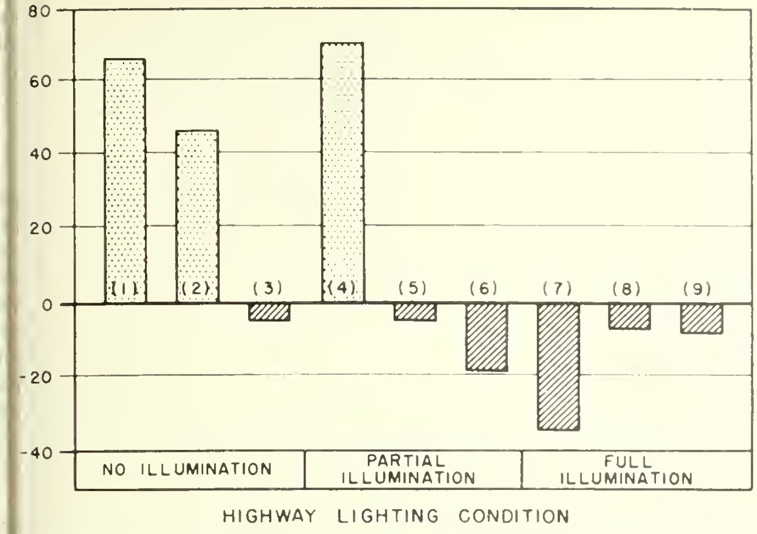
For ease and accuracy in recording observ
data, the acceleration and deceleration lan
were divided into three equal sections. T
percentage of total vehicles crossing into a
out of each third of the lane under each lighti
condition at night was paired with the co
parable percentage figure for the average da
The difference between the figures was th
expressed as a percentage of the average d
figure, and the percentages for the three se
tions of the speed-change lanes were average
The averages of the 3 percentage variatio
from the average day are plotted in figures
and 14 for each lighting condition. No p
centage variation is given for condition No.
as no data for "lane lines only" was availab
The average variation is in effect a measure
how close night operation approaches dayti
operation.

For site 4M, figure 13, it was found th
under the same conditions of delineation, u
of the acceleration lane at night was direct
related to the level of illumination. As t
amount of illumination increased, night usa
of the acceleration lane more nearly a
proached daytime usage. For example, wi
delineation and no illumination, condition N
3, night usage of the acceleration lane was
percent different from the average day; w
one-half illumination, condition No. 5, t
variation dropped to 23 percent; and with f
illumination, condition No. 9, the variatio
was 17 percent.

Table 4.—Summary of average speed, lateral placement, and clearance for light vehicles in through traffic lanes

Highway lighting condition number and code [1]	Site 4M						Site 5M					
	Speed		Placement [2]		Clearance [3]		Speed		Placement [2]		Clearance [3]	
	Day	Night	Day	Night	Day	Night	Day	Night	Day	Night	Day	Night
	m.p.h.	*m.p.h.*	*Feet*	*Feet*	*Feet*	*Feet*	*m.p.h.*	*m.p.h.*	*Feet*	*Feet*	*Feet*	*Feet*
No illumination:												
1.—LL	49.8	47.0	6.6	6.1	6.9	6.2	55.9	57.2	6.4	6.0	8.6	8.0
2.—L-EL	51.3	48.1	6.7	6.8	7.4	6.8	56.8	56.5	6.7	6.5	8.0	7.3
3.—L-EL, D	49.2	47.2	6.7	6.4	7.4	7.0	58.5	57.7	6.8	7.2	8.3	8.3
Partial illumination:												
4.—L-EL, ½I	50.3	51.2	6.9	6.4	7.4	6.2	57.5	58.4	6.8	6.9	8.4	8.0
5.—L-EL, D, ½I	50.8	50.0	6.5	6.5	7.1	7.8	56.9	55.5	6.6	6.7	8.1	8.5
6.—L-EL, D, ½I (int.)	49.8	50.5	6.6	6.6	7.2	6.8	57.4	56.7	6.8	6.6	8.1	7.9
Full illumination:												
7.—L-EL, D, I (int.)	48.8	49.7	6.5	6.6	7.1	7.6	58.9	58.3	7.1	6.7	8.4	8.2
8.—L-EL, I	51.0	49.0	6.9	6.5	8.3	7.6	59.1	58.5	6.9	6.9	8.2	8.4
9.—L-EL, D, I	49.5	49.8	6.7	7.0	7.6	7.3	57.8	57.5	7.2	6.8	8.4	7.9
Average	50.0	49.2	6.7	6.5	7.4	7.0	57.6	57.4	6.8	6.7	8.3	8.1

[1] For lighting code, see table 1, footnote 1. [2] Distance in feet center of car to right edge of lane. [3] Clearance in feet between bodies of adjacent cars.



gure 9.—Night from average day percentage variations of speeds for through light vehicles traveling below 40 miles per hour under the nine lighting conditions at site 4M.

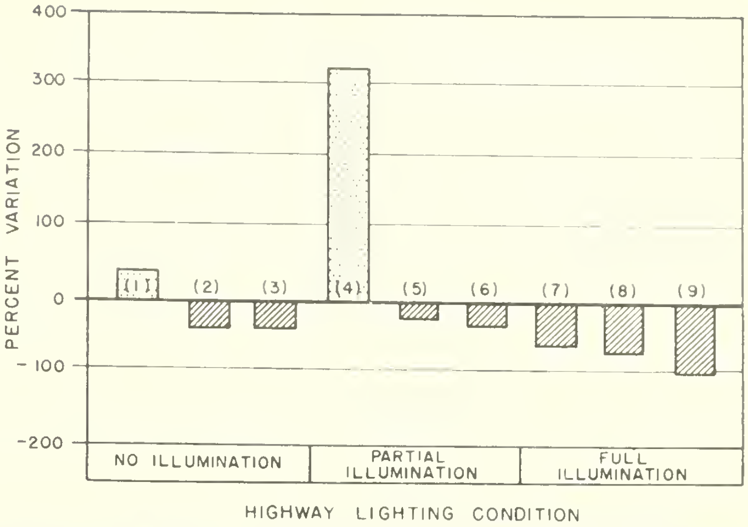


Figure 10.—Night from average day percentage variations of speeds for through light vehicles traveling below 40 miles per hour under the nine lighting conditions at site 5M.

In the use of the deceleration lane at site I, figure 14, full illumination also yielded e best results. One-half illumination, on e average, produced slightly larger variations m daytime operations than no illumination. Again the importance of delineation was monstrated. Under full illumination with lineation, the percentage variation was alst negligible, whereas under full illuminan with no delineation the variation was er 15 percent.

Other driver behavior recorded included e of brakes, sudden deceleration, cutting er, and changing lanes. Analyses of the erved data revealed nothing of significance. ere was more sudden deceleration and use of akes at the exit ramp site for condition No. lane and edge lines only, than under the ner lighting conditions during hours of rkness. During daylight hours, there was even higher total of such driver behavior both study sites.

cident data analyzed

Correlation of traffic operations with accits was found to be rather inconclusive bese of the limitations of the data available. Traffic volumes and other highway characteristics were different for the illuminated and nonilluminated sections of the Connecticut Turnpike. Incomplete roadway openings, general construction cleanup, etc., made it necessary to limit the surveillance of Turnpike accidents to the first 8 months of 1959.

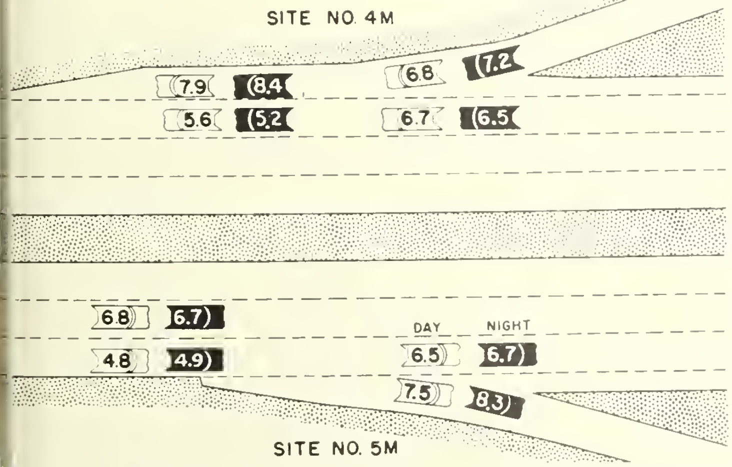
A summary of the general accident statistics for the illuminated and nonilluminated sections of the Turnpike is shown in table 5 along with similar data for the nonilluminated Merritt Parkway and all State highways.

The accident rates for both day and night travel on the Turnpike were considerably lower than on either the Merritt Parkway or the State highways. This was to be expected because of the higher standards of design of this modern controlled-access facility and its traffic appurtenances.

Analyses of the accident rates on the Turnpike revealed a slightly higher rate for the illuminated section during the day than for the nonilluminated section, and an appreciable higher rate for the illuminated section during the nighttime. Comparing day and night accident rates on the illuminated section, the night rate was 1.76 times that of the day rate; while on the nonilluminated section, the night rate was 1.36 times the day rate. However, testing the differences statistically, the accident rates did not prove to be significant. Also, when evaluated with respect to the wide variation in volumes and other characteristics between the illuminated and nonilluminated sections, the differences between the accident rates did not appear to be significant.

Analyses were also made of the relative exposure to accidents. This basically involved a comparison of the traffic volumes, frequency of ramp intersections, and general roadway features. While it appeared that conditions in the illuminated section were considerably more conducive to accidents than in the nonilluminated section, the following distinctions were made:

- The general roadway design standards in the two sections were the same for maximum curvature, maximum gradient, and ramp intersections.
- The average daily traffic volume on the illuminated section was approximately 3.6 times that on the nonilluminated section.



ure 11.—Average day and night lateral placements for light ehicles, averaged for all lighting conditions. Distance in feet rom center of vehicle to right edge of traffic lane.

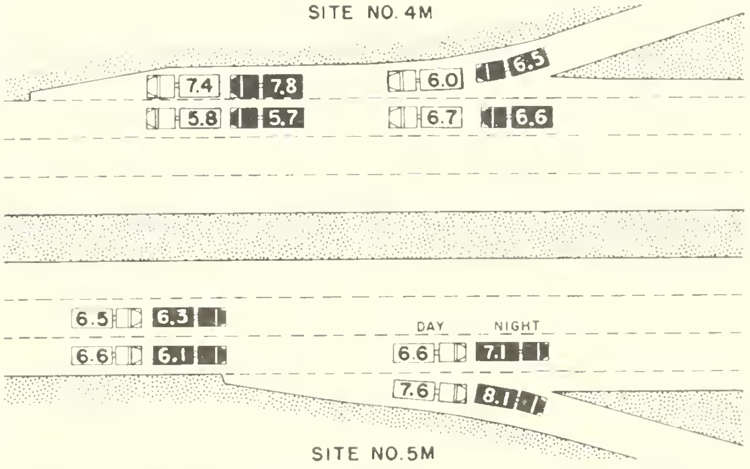


Figure 12.—Average day and night lateral placements for heavy vehicles, averaged for all lighting conditions. Distance in feet from center of vehicle to right edge of traffic lane.

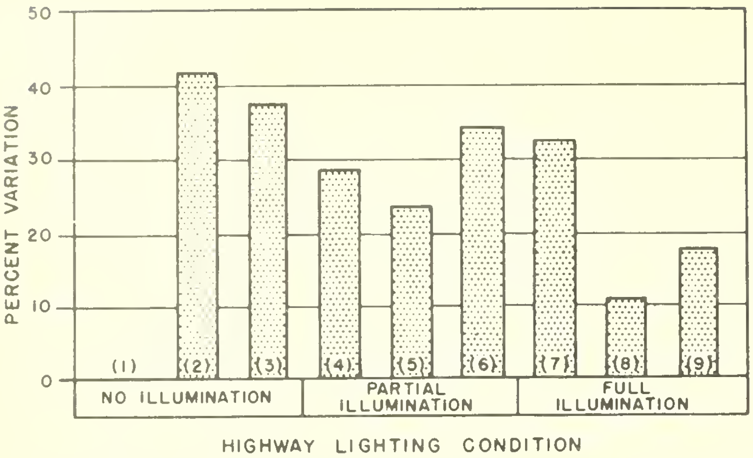


Figure 13.—Night from average day percentage variations of use of acceleration lane at site 4M. (No data available for condition No. 1.)

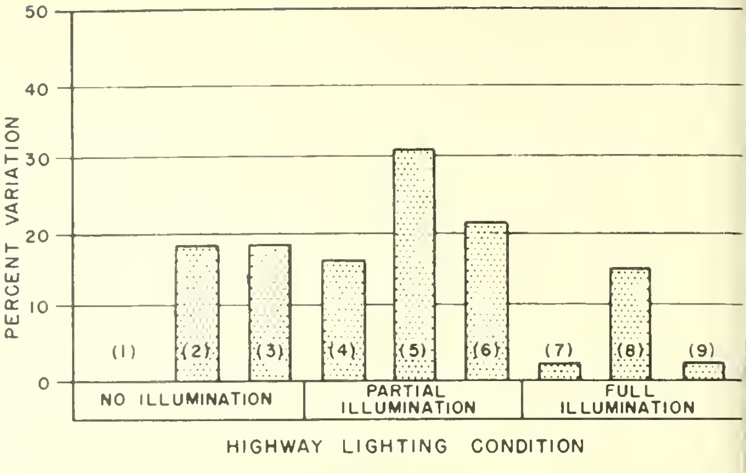


Figure 14.—Night from average day percentage variations of use deceleration lane at site 5M. (No data available for conditio No. 1.)

- There were twice as many ramp intersections per mile along the Turnpike in the illuminated section.
- The illuminated section followed a more undulating course than the nonilluminated section.

The importance of these highway and traffic features was evidenced by the fact that within the illuminated section, 43 percent of the accidents occurred in only 27 percent (14.3 miles) of the total 53 miles. The average daily traffic volumes in the 14.3-mile section ranged from 30,000 to 40,000. A portion of the 14.3-mile section, the 6.9 miles with the highest traffic volume, which was about 1.6 times that for the total 53 miles, was also subjected to a comprehensive examination. The comparative results are shown in table 6.

Several significant facts were evident from the accident summary. The daytime accident rate of the 6.9-mile section was 114 per 100 million vehicle-miles, or 1.54 times the daytime rate (74) for the entire 53 miles of illuminated highway. The nighttime accident rate of the 6.9-mile section was 135 per 100 million vehicle miles, or 1.18 times that of the daytime rate and only 1.04 times the average nighttime rate for the 53-mile section.

The daytime accident rate increased with the traffic volumes whereas the nighttime rate remained substantially the same and was then only slightly greater than the daytime rate on the same section. It must be realized that in the areas under discussion the traffic volumes during the day were about double those during the night.

It would appear from the analyses of accident rates for the illuminated sections of varying traffic volumes that highway illumination may have had a beneficial effect on accident experience. However, it was evident that the value of accident experience in establishing criteria for highway illumination would require considerably more da than was available, if statistically significa results were to be obtained.

Table 5.—Summary of accident data for the Connecticut Turnpike, Merritt Parkway, and other Connecticut State highways

Facility	Highway mileage	Total travel, vehicle-miles		Number of accidents		Accident rate[1]		
		Day	Night	Day	Night	Day	Night	Ratio: night/day
		100 million	*100 million*					
Connecticut Turnpike:[2]								
Illuminated	53.0	2.40	1.29	177	168	74	130	1.76
Non-illuminated	75.7	.99	.53	71	52	72	98	1.36
Total	128.7	3.39	1.82	248	220	73	121	1.66
Merritt Parkway—1957	37.5	3.45	.97	193	402	143	414	2.90
All State highways—1957	3,224.7	39.07	16.75	8,736	6,117	224	365	1.63

[1] Number of accidents per 100 million vehicle-miles.

[2] Accident data from Jan. 1, 1959, to Sept. 1, 1959.

Table 6.—Accident summary of the high traffic volume areas in the illuminated section of the Connecticut Turnpike

Highest traffic volume sections	Total travel, vehicle-miles		Number of accidents		Accident rate[1]	
	Day	Night	Day	Night	Day	Night
	100 million	*100 million*				
6.9-mile section	0.44	0.23	50	31	114	135
14.3-mile section	.85	.45	80	70	94	156
Total illuminated length (53 miles)	2.40	1.29	177	168	74	130

[1] Number of accidents per 100 million vehicle-miles.

Summary

The results of the analyses of the stu data presumably can be considered typic for a freeway similar in design and traf volumes to that of the Connecticut Turnpik The interchange studied was probably al typical of freeway interchanges both fro the standpoint of geometrics and traf volumes. Under congested traffic condition the results might differ considerably.

The more important results evolved fro the study are as follows:

1. Neither average speed, lateral placeme headway distance, nor lateral clearan showed any consistent change between d. and night conditions by virtue of highw illumination or delineation.

2. Nighttime usage of the acceleration la approached daytime usage as the level illumination increased. A similar patt existed at the deceleration lane, although was more variable.

3. In general, it appears that benefic results of illumination in the decelerati area were derived when it was used at the f level and that even greater services w derived when illumination was combined w roadside delineation. Illumination of t interchange area only did not appear to desirable insofar as the access ramp site v concerned.

4. Analyses of the accident data for illuminated and nonilluminated sections f the Connecticut Turnpike do not prov conclusive results because of the extre variance in traffic volumes and other highw characteristics.

Despite the extensive analyses of tremendous volume of data recorded, positive or significant trends could be de oped that would define specific warra for highway lighting. However, the res of the investigation did point up the va

(Continued on page 71)

Digital Recording for Highway Research

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported [1] by RICHARD C. HOPKINS,
Chief, Instrumentation Branch

THE moving finger of Omar Khayyam possibly could have been inspired by the contrivance of some 12th century scientist who attached a stylus to a lever transducer so that an ancient analogue was produced on a clay or wax tablet drawn through this device by a slave walking in cadance with a water clock. The discoveries of magnetism and electricity did not produce any changes in this basic system; they only brought operational mprovements such as pen motors and galvanometers, capillary pens and light beams, spring clocks and synchronous motors, and paper charts to replace the limited clay tablet.

Since the inception of the oscillograph, housands and thousands of miles of analogue charts have been recorded, analyzed, transcribed, and filed. And the analogue chart, which may be described as a pictoral represenation of a fact, will certainly always be with us; but, its future use will become more and more confined to special and carefully selected applications. Taking the place of the anaogue, digital recording (recording information s numbers) is becoming more commonly used and appreciated by the research engineer.

Automatic digital recording was possible nd could have been in common use several ears ago. However, the data-reduction procsses were limited to the use of desk calculatng machines or punched card machines, and t was relatively easy for clerks to supply igits from analogues as fast as calculating-machine operators or key punch operators ould handle them.

Only a few short years ago, the digital omputer, in the evolution of electronics, was hanged from a million-dollar wonder to few-thousand-dollar office machine. Users oon became aware of the machine's tremenous capacity for digits. It was able to ingest nd digest digits faster than small armies of 0th century clerks could analyze, transcribe, nd key-punch analogue charts.

The obvious answer to the problem of the lle computer was to eliminate the analogue nd to record all information intended for utomatic data reduction in digital form. So, tilizing the circuits and techniques that had een developed for the digital computer, ectronic instrument manufacturers have made available "hardware" and complete nits which will deliver digital records from xisting analogue systems in addition to perorming in their primary application in newly esigned digital systems.

Glossary of Terms

Some of the electronic terms employed in this article are as follows:

Print command.—An electrical signal which indicates to a digital recorder that the periodic data collection is complete and is to be printed. This signal may be either manually or automatically controlled.

Staircase.—A digital code in which each digit from 0 to 9 is represented by a discrete voltage that is one of equal sequential steps above or below a design reference voltage.

10-Line.—A digital code in which each digit from 0 to 9 is represented by an individual electrical contact.

Binary code.—A term referring to one of several electrical methods of indicating a decimal number in a binary system. The two most frequently used systems in digital recording totalize binary indications of 1-2-4-8 or 1-2-2-4 to obtain the desired digit.

Time base generator.—An electronic device which provides a repetitive signal of known frequency at a high degree of accuracy. (See digital clock.)

Digital clock.—An electronic device which accepts the output of a time base generator, accumulates totals of usable time intervals, and provides for the time increment recording in a digital system.

Electronic decade scaler.—A binary device which counts electrical input signals and provides a digital output of the total between 0 and 9. The device then produces a "carry-over" signal and repeats its counting sequence.

Digital voltmeter.—An electronic instrument which can accept direct current voltages and display the values as decimal numbers. The voltmeter also provides for a recorder input by one of the standard digital codes.

Digital Recording Systems

The purpose of this article is to introduce those familiar with analogue recording techniques to some of the standard electronic units which may be combined to form flexible digital recording systems.

The Bureau of Public Roads has been digitally recording traffic survey data since 1946 when special solenoids were mounted on standard adding machines due to the lack of commercially available equipment. Ten years later it was still necessary for the Bureau of Public Roads to provide its own solenoids. Today, however, the manufacturers of business machines provide for remote electrical actuation of nearly all models and almost all manufacturers either provide for a direct output to a motorized tape punch or have an auxiliary unit to provide this output. This rapid trend toward the remote or automatic handling of data by office machines is only mildly indicative of the advances being made in electronic computer print-outs or recorders designed for digital systems.

These newer recorders will be briefly discussed and a few typical systems will be shown that can be used individually or in combination to produce direct printed records of measures of physical quantities.

It is difficult to observe any digital system without becoming fascinated by the rapidity and positiveness with which the data are printed and to admire the complex mechanism which so faithfully transfers electrical intelligence into numerals. In fact, the effect is to often overweight the importance of the recorder which may mask a much more complicated system of data collection. So,

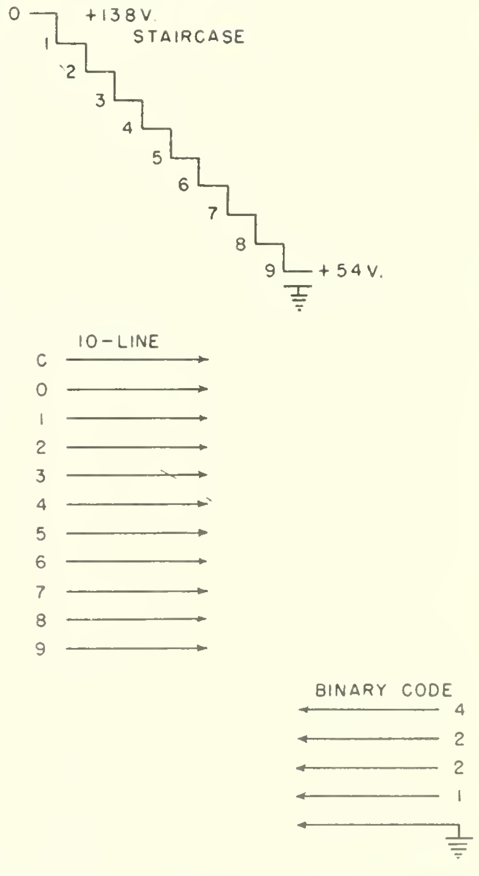


Figure 1.—Three digital recorder input codes.

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[1] This article was presented at the 39th Annual Meeting the Highway Research Board, Washington, D.C., January 60.

because of the straightforward design of these recorders, it is possible for the purpose of this article to consider only a few external features that are pertinent to any digital system and then consider in more detail such systems and possible alternates.

Selection of the first digital recorder will probably be dictated by the output of the system that has determined its need. Therefore, since the features of accepting and storing digits to be printed on command are common to all recorders, it is only necessary to refer to recorders in terms of *staircase*, *10-line*, or *binary code*. These digital recorder input codes are illustrated in figure 1.

In a *staircase* input each digit in each column is represented by some particular voltage with respect to a zero level. A typical example is the use of +138V to represent "0," +54V to represent "9," and the other digits represented by voltages equally separated between these two.

The *10-line* recorder provides a common terminal for a battery connection and 10 digit terminals for each column. A digit is stored in such a recorder by simply completing the circuit from the other terminal of the battery to the proper digit terminal.

A *binary code* input is what the name implies. In this system, the sum of the code units is the decimal equivalent of the binary number produced by the electronic system. Because of electronic expediency, code values of 1–2–2–4 are usual although recorders with a 1–2–4–8 input are available.

In physical dimension, a digital recorder is about 20 inches long, 12 inches high and 15 inches deep and may be obtained in an individual cabinet or with a panel for relay rack mounting. The recorder normally operates from 120V 60-cycle power with a demand of the order of 150 watts. Recorder models are available to print from 6 to 12 digits as rapidly as 5 times per second with some special computer print-outs capable of much higher recording rates.

Time recording

In considering the systems that collect and prepare the data to be recorded, the measurement of time, being the abscissa of almost all of the common analogue records, should probably be considered first. Unit time in the analogue is unit length and its measure is dramatically illustrated by the bulk of the recorded chart.

In a digital system, time, like all other quantities, must be recorded in its numerical value. An exception to this would be a situation which permitted the recorder print command to be controlled by time units. Each record would then indicate the passage of one time unit and clapsed time could be determined in the data reduction process by totaling the individual recordings.

In the general case, however, a time base generator or digital clock must be included as a part of the data collection system and its significant figures recorded. Two typical time bases are shown in figure 2. A quartz crystal oscillator (usually 100 kc.) is connected to suitable electronic frequency divider circuits to produce the time unit required. These time units are counted by an electronic decade scaler with an output for each count of 10 carried over to another similar scaler. As many decade scalers as necessary may be interconnected and the output (staircase or binary code) of each scaler is always available for transfer to a proper digital recorder.

A second practical, and somewhat more mechanical, system would utilize a synchronous motor, proper reduction gears, and an electrical contact closed by the driven cam. Ten contact, spring-driven stepping switches would then be used in each decade and a 10-line output to a digital recorder would be available.

As circumstances might require, the stepping switches could be actuated by the quartz crystal oscillator and divider circuits or the electronic decade scalers could count the contact point closures. Other suitable time base circuits make use of capacitor discharg time, tuning forks, direct counting of th powerline frequency, etc.

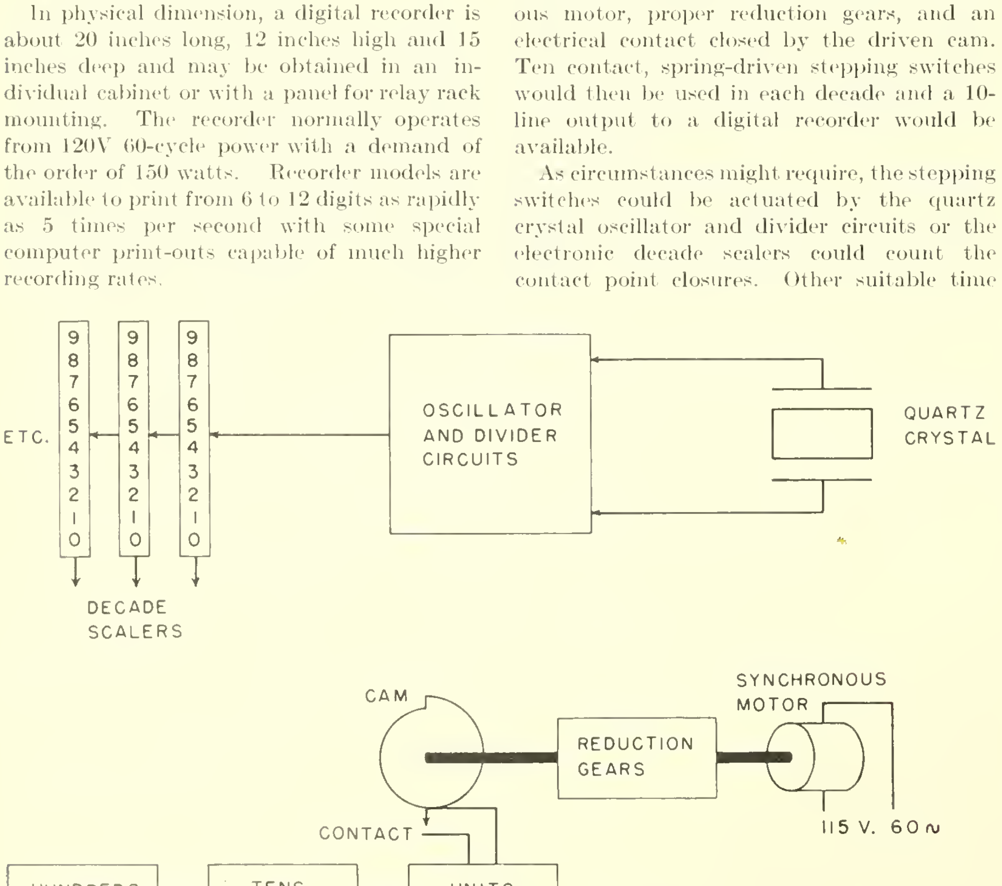


Figure 2.—Typical digital clocks for time recording.

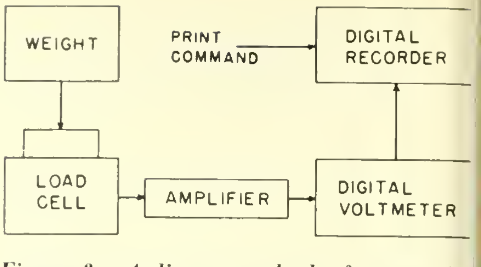


Figure 3.—A direct method of convertin weight analogue systems to digital re cording.

Weight recording

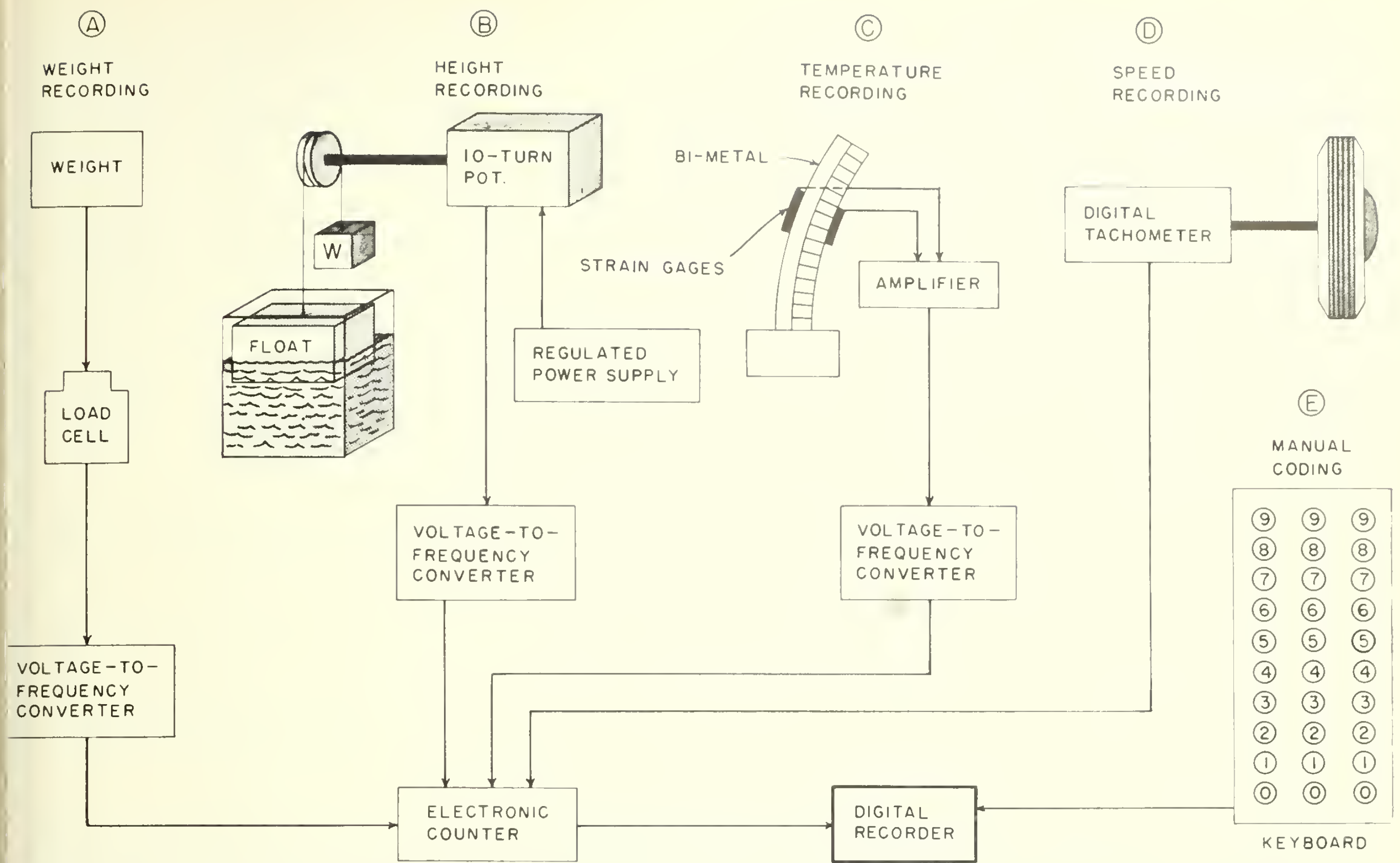
Weight can be directly detected by loa cells. For use in conjunction with direc writing oscillographs, load cells and th necessary oscillator-amplifier systems ar commonly used in many highway researc laboratories. Two alternative methods c converting typical laboratory weight analogu systems to digital recording are possibl Both are equally accurate and the choice wi be dictated by future utilization of th equipment which is to be procured.

A direct substitution approach, figure 3, to simply replace the direct writing oscill graph with a digital voltmeter. For a mi mum of data collection, the weight values ca be manually recorded from the digital indic tion on the voltmeter. However, since it assumed that large quantities of data are t be collected and that a digital recorder h also been obtained, the digital voltmeter w be directly connected to a compatible digit recorder and automatic data collection will l accomplished on each print command.

The less direct method, but in mar respects more desirable, diagram A in figure is to substitute a voltage-to-frequency co verter for the direct writing oscillograph. electronic counter unit is also added and used to measure the output frequency a transfer the weight data to the digital record Although requiring an additional unit, t method is favored because the counter u will have many more laboratory applicati than a digital voltmeter and the origi extra expense will result in later economi Furthermore, the two units will always co bine for a direct measurement of voltage.

Vibration or dynamic strain recording

Vibration or dynamic strain investigati have encouraged the purchase of much of t electronic equipment now used in highw and structural laboratories. The usual s tem to record vibration or dynamic strain standard carrier-amplifier equipment w either a direct-writing or galvanometer-t oscillograph output. The digital system these data recording, as shown in figure requires a direct measurement of two valu amplitude and frequency. The amplitude detected by a peak voltmeter for measurem by a standard digital voltmeter and the f quency is directly determined by one of standard electronic counters. Both ampliti

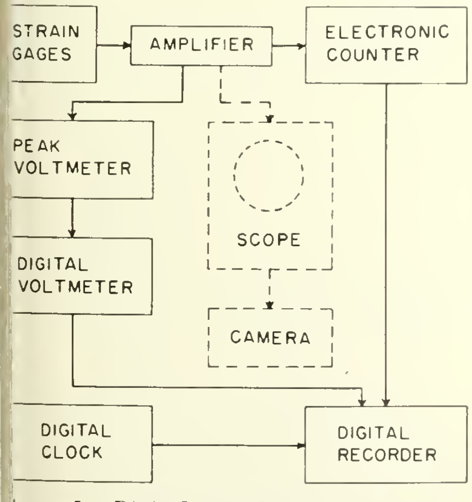


igure 4.—Digital recording diagrams for weight, height, temperature, and speed (and electrical keyboard for coding) using the voltage-to-frequency converter and the electronic counter.

id frequency can then be recorded as desired id the time saved can be best appreciated y those who have counted cycles on analogue iarts to determine frequency. The addition a cathode ray oscilloscope will permit the bservation of waveform and a camera will rovide the few necessary illustrations for chnical reports.

eight recording

Liquid levels, large displacements in oper-ing mechanisms, and other height recordings, can be measured and recorded by the system shown as diagram B in figure 4. A precision, 10-turn potentiometer divides the voltage output of the regulated power supply in direct ratio to the level being measured. The print command would usually be controlled by some digital clock and a direct printed record is the result.



igure 5.—Digital recording of vibration or dynamic strain.

Deflection recording

Figure 6 shows a system somewhat similar to that used in recording height but which is more practical for small displacements or deflections. In this system a linear precision potentiometer, or a linear variable differential transformer (LVDT) in a carrier-amplifier system, provides a direct reading of movement. The print command in this instance could be controlled by position detectors "D" to provide a deflection record of a structural member.

Temperature recording

Any system of temperature detection with an electrical output can be adapted for digital recording. Diagram C in figure 4 shows one typical system in which strain gages are attached to a bimetal strip. A standard bridge-amplifier couples such a detector to a digital voltmeter for transfer to the recorder at any desired time. Other detectors, such as thermistors or thermocouples, can be easily accommodated.

Speed recording

Shaft speeds are detected by a tachometer as in analogue systems. However, for digital recording it is generally more convenient to use a digital tachometer rather than converting an analogue output. The common tachometer generator produces an increasing voltage with increasing shaft speed; the digital tachometer produces one or more pulses per revolution of the shaft. The digital recording system is completed by connecting a digital tachometer to an electronic counter which has a time base and gating circuits to count pulses per unit of time. The shaft speed then

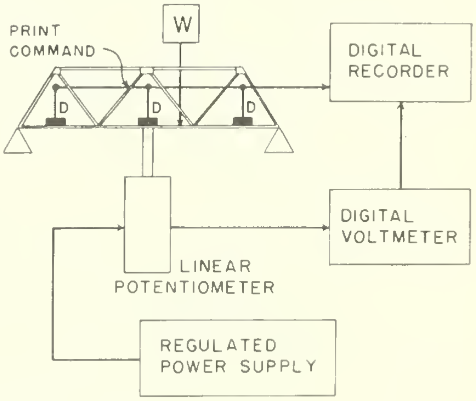


Figure 6.—Digital recording of small displacements or deflections.

becomes direct reading for transfer to the digital recorder. A typical system is shown as diagram D in figure 4.

Manual coding

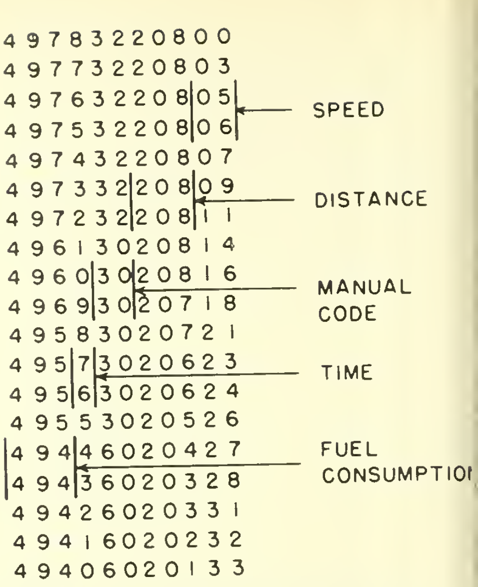
Notations, which must be entered by hand on an analogue chart, can usually be more conveniently handled by an electrical keyboard in a digital system by entering numerical codes directly in the recorder. This can always be done by providing decades of push buttons connected to produce the proper digital code for the type of recorder used (diagram E in figure 4).

Traffic Impedance Analyzer

It is a rare occasion when a physical investigation requires the recording of only a single channel of information. It is much more likely that the capacity of any given recorder will be strained and a complicated system for programing the data entries will become desirable. However, most problems can be reasonably limited to the capacity of the 11- or 12-digit recorders available.

An example of several data being simultaneously entered on a digital recorder is provided in the *traffic impedance analyzer* developed by the Bureau of Public Roads for speed and delay studies and operating economy studies. This digital data-collecting and recording system is mounted in a motor vehicle, figure 7, and when the vehicle is in motion, at 1-second intervals, prints the vehicle speed in miles per hour, accumulated mileage in three significant figures, time in seconds for a sequence check, and three significant figures of fuel consumption, and provides 2 decades of manual code. These data are shown in a typical recording sample in figure 8.

Figure 9 presents a block diagram illustration of the traffic impedance analyzer and simplifies the explanation of its operation. As both speed and delay studies and operating economy studies require speed and distance to be recorded as a part of the data, primary consideration was given to recording these quantities. The speedometer cable of standard American automobiles is designed to make 1,000 revolutions per mile. On this basis a special digital tachometer was designed which would provide direct speed and distance outputs. For the measurement of speed, a disc with 36 holes near its outer edge was rotated by a shaft input. With a light source on one side of the disc and a photoelectric cell on the other, electronic pulses in direct representation of speed in miles per hour could be generated each one-tenth second with a shaft input rate of 1,000 revolutions per mile. A second disc with one hole near its outer edge was coupled to the first disc by a 10:1



```
4 9 7 8 3 2 2 0 8 0 0
4 9 7 7 3 2 2 0 8 0 3
4 9 7 6 3 2 2 0 8|0 5|
4 9 7 5 3 2 2 0 8|0 6|
4 9 7 4 3 2 2 0 8 0 7
4 9 7 3 3 2|2 0 8|0 9
4 9 7 2 3 2|2 0 8|1 1
4 9 6 1 3 0 2 0 8 1 4
4 9 6 0|3 0|2 0 8 1 6
4 9 6 9|3 0|2 0 7 1 8
4 9 5 8 3 0 2 0 7 2 1
4 9 5|7|3 0 2 0 6 2 3
4 9 5|6|3 0 2 0 6 2 4
4 9 5 5 3 0 2 0 5 2 6
|4 9 4|4 6 0 2 0 4 2 7
|4 9 4|3 6 0 2 0 3 2 8
4 9 4 2 6 0 2 0 3 3 1
4 9 4 1 6 0 2 0 2 3 2
4 9 4 0 6 0 2 0 1 3 3
```

SPEED
DISTANCE
MANUAL CODE
TIME
FUEL CONSUMPTIOI

Figure 8.—A typical recording sample fron the traffic impedance analyzer.

gear reduction unit. A second photoelectri cell on the outer side of the second disc there fore generated an electronic pulse for eac one-hundredth of a mile of vehicle trave The shaft input of this digital tachometer wa coupled to the regular speedometer cabl of the automobile by a 1:1 geared "T" a the speedometer head.

The electronic pulses from the speed de tecting photoelectric cell were connected t the input of an electronic counter. Th counter included a quartz crystal time ba and generated usable pulses at intervals ($\frac{1}{10}$, 1, and 10 seconds. The $\frac{1}{10}$-second puls were utilized internally to operate the *cou*

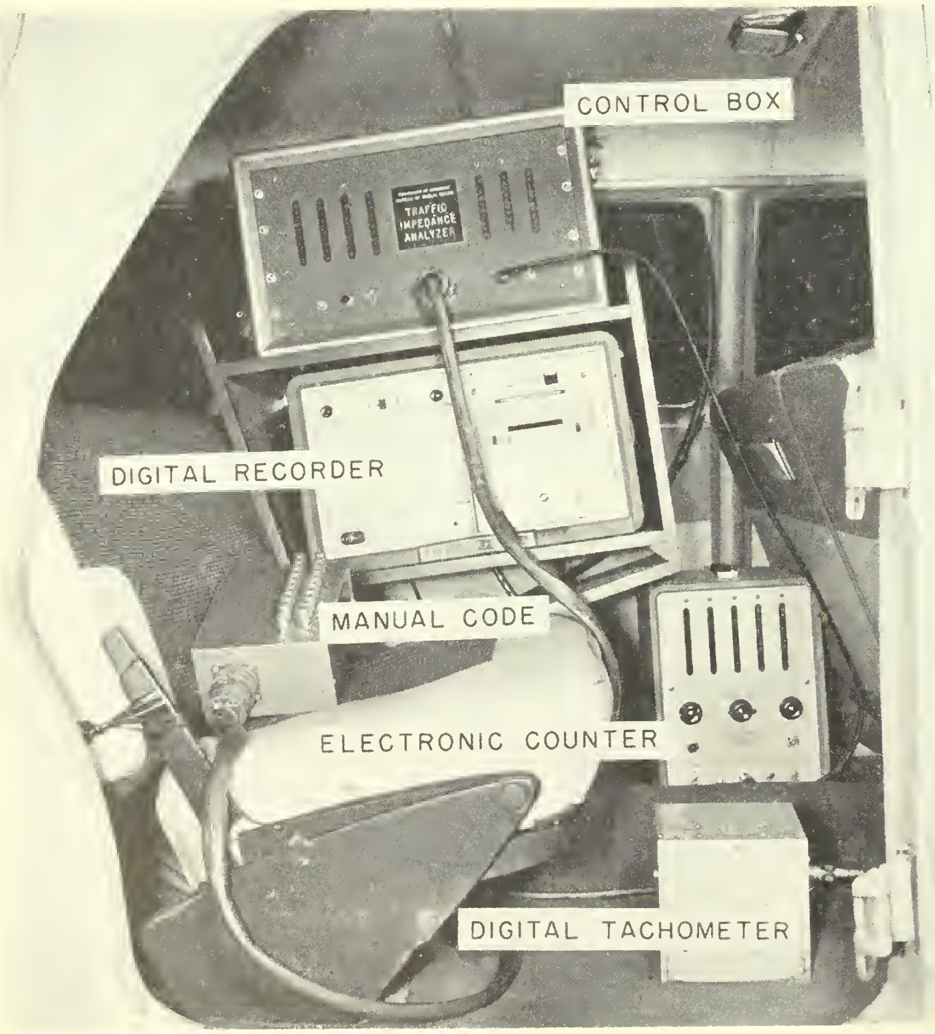


Figure 7.—Traffic impedance analyzer mounted on rear seat of a passenger car.

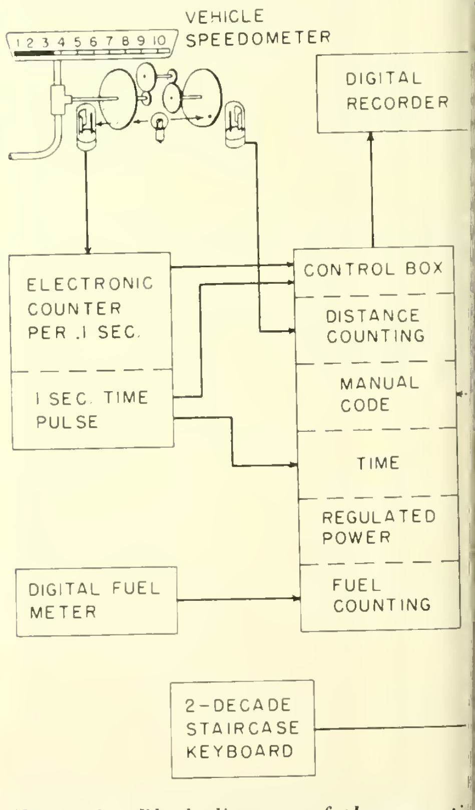


Figure 9.—Block diagram of the operati of the traffic impedance analyzer.

ınd *stop* gates so that the number of *speed* ɔulses for any $^1/_{10}$-second period could be ıccurately counted. The 1-second pulses vere connected to the control box and used to nitiate a recording period. Therefore, at the ɔeginning of each second the electronic ounter was reset and the *count* gate opened o accumulate *speed* pulses. One-tenth secınd later the *count* gate was closed and a ɔrint command recorded the speed of the ehicle and all other data which were available at the recorder inputs.

The electronic pulses from the distanceetecting photoelectric cell were connected to decades of electronic counting in the control ox where they were accumulated. This istance accumulation was continually available ɔr recording.

The traffic, geometric design, control, eather, and other pertinent data could be oded by 2 decades of push buttons which would transfer as digits to the recorder at any printing cycle.

One decade of time counting was provided in the control box as a printing sequence check. The circuitry also provided for time counting to be extended to 4 decades as an alternative to fuel recording when it was more convenient, for field observation, to have elapsed time recorded during speed and delay studies.

To record fuel consumption a digital fuel meter was mounted on the engine of the motor vehicle between the fuel pump and carburetor. The digital fuel meter used in this instance produced an electrical pulse for each $^{1}/_{2000}$-gallon of gasoline passing through it. These pulses were accumulated in 3 decades of electronic counters in the control box so that a total of fuel consumption was available for transfer to the recorder at each print command.

The complete equipment was powered by a 1,250-watt gasoline-driven generator weighing 85 pounds which was carried on a standard roof-mount luggage rack. This self-contained unit illustrates the advantages of digital recording and the possibilities of adapting a few standard electronic instruments to the myriad data collecting problems encountered by the highway research engineer.

Fully automatic data reduction can, of course, be realized by adding standard tape punch units to the digital recorders discussed herein. The punched tape then may be used as a computer input or its data may be automatically transferred to magnetic tape or punch cards as individual cases require. The equipment to accomplish these processes are available to those who may wish to extend their systems to include them; but, for the purposes of this article, it was believed that a simple introduction to digital recording would better serve the greater number of highway engineers.

Driver Tension Responses

(*Continued from page 58*)

Not only did the observation program iminate temporal differences, it also elimıated intensity differences within each traffic vent. It was assumed that all occurrences an event represented the same stimulus tensity. In essence, this procedure elimates within stimulus class variability. It is ıite obvious that the affective intensity of ı event may be quite variable, depending ɔon a variety of temporal and spatial factors the environment as well as perceptual and notional factors in the driver. It should be ɔvious, therefore, that the method used in is study oversimplified the nature of the affic interferences and simply placed them to 1 of 8 qualitative categories, none of hich had any measure of intensity attached. ıus, precision of stimulus measurement was crificed to obtain reliability of observation.

Within the limitations outlined, this study dicates that the two arterial routes studied differ in the rate and magnitude of galvanic skin response aroused in driving. The results, in general, indicate that the GSR may be a promising means for using driver behavior to discriminate between different kinds of streets and the interferences on them. There are, however, several statistical and methodological problems inherent in the use of the GSR which requires further research and which, consequently, restrict its operational utility at the present time.

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Highway Illumination and Delineation

(*Continued from page 66*)

roadside delineation with or without uınination, and the need for using adequate tensity of illumination if highway lighting to be provided. Above all, the experience ined will be most helpful in the planning d conducting of future highway lighting search, even if only in negative fashion. It appears that criteria other than those used in this report must be studied to evaluate properly the effect of highway lighting on traffic operations.

Editor's note: The detailed tables containing the schedule of field operations and the average speeds, placements, headways, and clearances for all motor vehicles under the nine conditions of illumination and delineation, are not published here since they presumably are not of interest except to researchers in this subject. Copies of the tables are available to those who may have specific use for them from the Division of Traffic Operations, Bureau of Public Roads, Washington 25, D.C.

Introduction to Highway Hydraulics: A Motion Picture

The Bureau of Public Roads, U.S. Department of Commerce, in cooperation with the Colorado State University, recently produced the motion picture, *Introduction to highway hydraulics*. This film illustrates basic hydraulic principles related to open-channel flow design in highway engineering.

Flow illustrations of the hydraulic principles are shown by laboratory models and field scenes. Various scenes depict the principles of continuity of flow, energy, and momentum, which are the basic laws governing any open-channel flow problems. Use is made of field scenes to illustrate where the principles apply in highway engineering.

Introduction to highway hydraulics is a 16mm. sound and color film with a running time of 21 minutes. Prints may be borrowed for showing by any responsible organization by request addressed to Mr. Ray B. Dame, Head, Photographic Section, Bureau of Public Roads, Washington 25, D.C. There is no charge other than for express or postage fees. Request should be sent well in advance of the desired showing, and alternate dates indicat if possible. Immediate return after ea booking is necessary, so that all requests ma be fulfilled.

Prints of the film may be purchased $86.47 per copy, the price including film, re can, shipping container and postage withi the United States. Inquiries on purchasin film prints should also be addressed to Mr. Ra B. Dame, Head, Photographic Section, B reau of Public Roads, Washington 25, D. Payment should not be sent with the inquir

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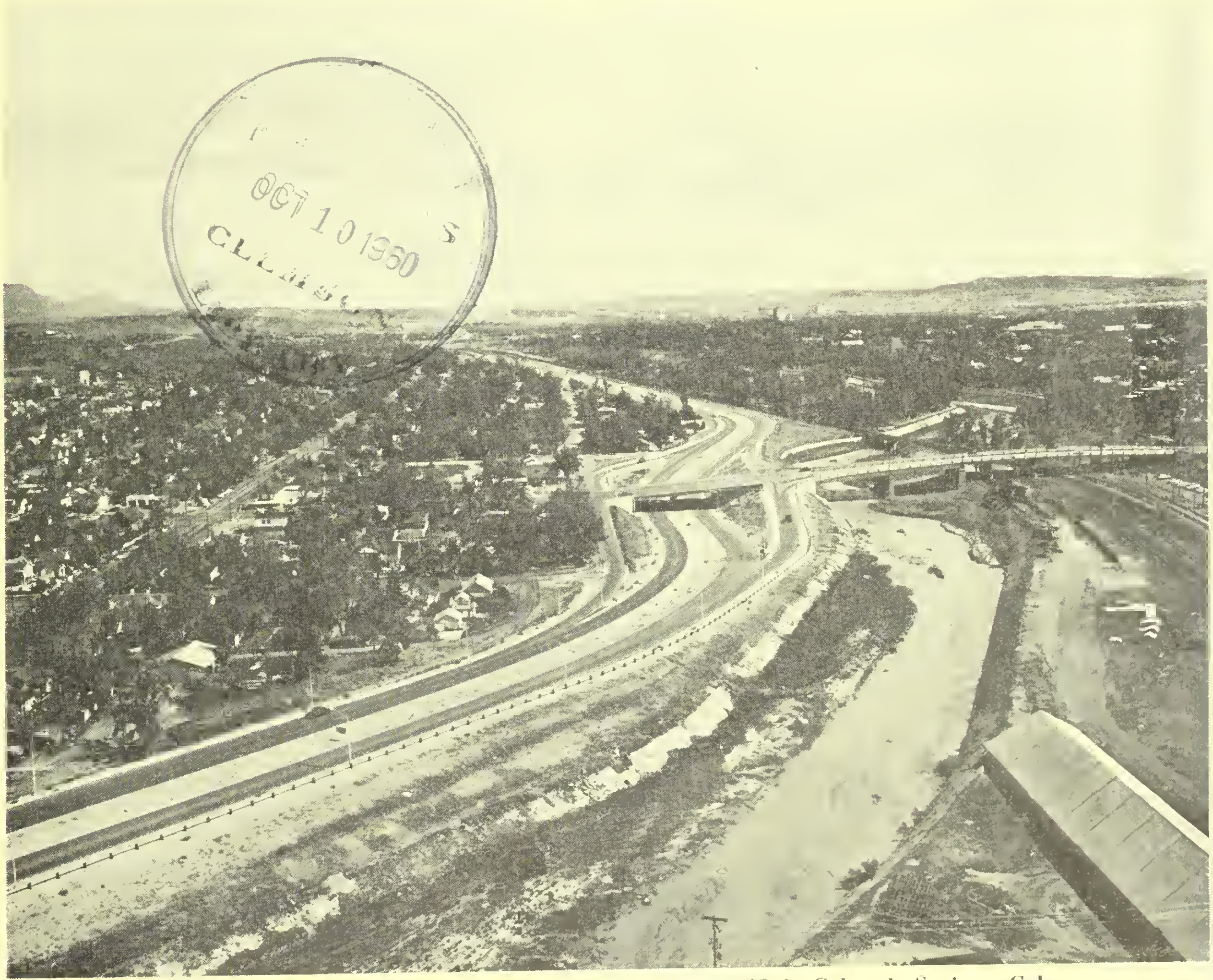

Monument Valley Freeway, Interstate Route 25, in Colorado Springs, Colo.

IN THIS ISSUE:

A discussion of properties of highway asphalts.

A new O-D survey technique—*continuous sampling.*

Public Roads

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Contents of this publication may be reprinted. Mention of source is requested.

Properties of Highway Asphalts—Part II, Various Penetration Grades

Y THE DIVISION OF PHYSICAL RESEARCH
UREAU OF PUBLIC ROADS

Reported[1] by J. YORK WELBORN, Chief,
Bituminous and Chemical Branch,
WOODROW J. HALSTEAD, Head,
Bituminous Materials and Chemical Section,
and J. GAYLOID BOONE, Analytical Chemist

This article is the second in a general study of asphalt cements produced in the United States for highway construction. A total of 323 samples from 105 refineries were received and tested. Results of the tests on 146 samples of the 85–100 penetration grade were reported in the first article, part I.[2] This article, part II, includes the results of the tests on the other penetration grades—namely, the 60–70 grade asphalts, the 70–85 grade, 120–150 grade, and 150–200 grade asphalts. Comparisons of the characteristics of several grades including the 85–100 grade reported in part I are also included.

These data establish the range for test values that may be expected for asphalts on a nationwide basis. The significance of the wide differences existing in materials meeting the same specifications, and the need for further research to establish more adequate test methods and specifications, are discussed in the article.

N 1954, a comprehensive study of asphalts was undertaken by the Division of Physical esearch to provide current data on a national ale that would show the properties of asphalt oduced from the various crude sources and the different methods of refining in current e in the United States. The samples were tained by the regional offices of the Bureau Public Roads with the full cooperation of e States and refineries in each area of the tion.

Nearly all producers supplying asphalt for e study included samples in the 85–100 netration grade. Since this group was nsidered the most representative of all phalt cements produced and used, it was lected for the initial study. The results of ose tests were reported in PUBLIC ROADS part I of the extensive study.[2] This ticle, part II, presents the results of the tests de on the other penetration grades.

A total of 323 samples were received during 54 and 1955 from 105 refineries or storage rminals. Of these, 146 were in the 85–100 netration grade which was reported in rt I. This report includes the results of sts on the following asphalt cements: 60–70 ade, 58 samples; 70–85 grade, 30; 120–0 grade, 60; and 150–200 grade, 16 samples.

It may be noted that the numbers of mples tested in each grade differ slightly om the numbers given in the first report. This is due to the elimination of a few samples because of the lack of proper identification of their origin. Also, since only 7 samples of the 100–120 grade were received, they have been omitted from the discussion.

Data in this report, as in the earlier report, include only those test characteristics in general use as specification requirements or those that are being used by some agencies in an effort to obtain better materials. All testing was performed according to ASTM or AASHO standard methods of test.

Tables 1 through 4 show the results of the tests for the 60–70, 70–85, 120–150, and 150–200 penetration grades, respectively. In each table refineries are identified by code number, the same number applying to the same refinery for all grades. To avoid possible confusion in sample identification all samples are numbered consecutively beginning at 120. Numbers 1–119 were those used for the 85–100 grade reported in part I.

Tables 1 through 4 also include information on the crude source and method of refining used to produce the asphalt cements. Although in many cases more precise information as to the geographical location of the crude sources and details of methods of refining were provided, only general terms are given in this report.

The 85–100 grade materials reported previously represent the majority of the samples submitted and is the grade most widely produced and used. In the earlier report of this penetration grade, frequency distribution graphs in the form of frequency polygons were used to show the range in properties. This report, part II, presents the data in a similar manner and directly compares the test results for the 60–70, 85–100, and 120–150 penetration grades. Since the number of asphalts received varied for the different grades, the percent of samples was used for the unit of the vertical axis rather than the actual number of samples. The percentages of values in each class interval were then plotted at the midpoint of the respective class interval and connected to form the polygon. Values exactly equal to the limit dividing two class intervals are included with the upper interval. Where they are believed to be significant, median values for the test results are shown on the graphs for purposes of comparison.

The data used for the graphs are the test values given in table 1 (for 47 of the 60–70 asphalts), table 3 (for 56 of the 120–150 asphalts), and table 1 of part I (for 119 samples of 85–100 penetration grade asphalts). Sample identification numbers appearing in tables 1 and 3 having a suffix a, b, or x represent duplications of materials from the same refinery and the tests values for these materials were not included in the analysis. It is recognized that the difference in representation of the number and source of samples in each grade may have some influence on the distribution polygons. However, since the samples were submitted as being representative of the asphalts used in the various States, the graphs should be generally indicative of properties of the available asphalts of these grades. Because the number of samples tested in the 70–85 grade and the 150–200 grade are limited and represent only a relatively small portion of the asphalts used in only a few areas of the Nation, they are not included in this analysis. However, the test data given in tables 2 and 4 should be of interest in those areas where such asphalts are used.

In order to obtain a direct comparison of the properties of the asphalt cements of different penetration grades from the same refinery source, some of the test data shown in tables 1–4 of this report and in table 1 of part I are repeated in table 5. The samples selected for this comparison are asphalt cements of three or more grades refined from the same crude

---

[1] This article was presented at the Association of Asphalt ving Technologists, Memphis, Tenn., January 1960.

[2] *Properties of highway asphalts—part I, 85-100 penetration de*, J. York Welborn and Woodrow J. Halstead, PUBLIC ADS, Vol. 30, No. 9, Aug. 1959, pp. 197-207.

Table 1.—Test characteristics of 60–70 penetration grade asphalts

B.P.R. region [1]	Sample identification [2]	Source of crude	Method of refining [3]	Penetration			Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Soluble in CCl₄	Standard oven test at 325° F., 5 hours			Thin-film oven test, ⅛-inch film at 325° F., 5 hours				
				100 g., 5 sec. at 77° F.	200 g., 60 sec. at 39.2° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute 77° F.	1 cm. per minute 39.2° F.				Pensky-Martens closed cup	Cleveland open cup		Loss [4]	Penetration of residue		Loss [4]	Tests on residue			
																Value	Percent of original		Softening point	Ductility, 5 cm. per min., 77° F.	Penetration at 77° F.	Percent of original penetration
							Cm.	Cm.	°F.	Sec.		°F.	°F.	Pct.	Pct.			Pct.	°F.	Cm.		
Region 1	120 (2)	Venezuela	V, S	66	24	36	250+	9.5	124	283	1.032	500	540	99.73	0.03	59	89	0.10	133	168	42	61
	121 (22)	Mexico	S	57	25	44	250+	11.3	128	431	1.039	480	545	99.84	.06	52	91	.29	140	100	37	65
	122 (30)	Colombia		64	22	31	187	10.0	121	247	1.011	500	640	99.57	.04	58	91	+.07	129	250+	47	73
	123 (32)	Venezuela	V	53	28	53	250+	6.0	131	342	1.018	490	565	99.84	.02	48	91	.07	142	47	35	66
	124 (77)	Venezuela, Texas	V, S, O	64	25	39	228	8.0	122	295	1.021	435	580	99.75	.18	53	83	.45	135	162	37	58
	125 (86)	Gulf Coast	V	62	25	40	250+	8.3	123	267	1.015	530	600	99.86	.00	55	89	.00	132	223	45	73
	126 (90)			52	22	42	250+	8.8	126	279	1.021	480	560	99.87	.07	48	92	.18	135	218	38	73
Region 2	127 (1)			64	23	36	250+	9.8	122	358	1.035	495	540	99.75	.01	57	89	.05	133	183	42	66
	128 (3)	Venezuela	V, S	60	25	42	250+	8.5	125	346	1.033	475	515	99.68	.09	51	85	.36	139	140	37	62
	129 (9)	do	S	64	23	36	215	24.0	126	347	1.038	455	505	99.80	.10	56	88	.46	137	113	38	59
	130 (14)			60	20	33	250+	10.5	122	213	1.024	500	575	99.92	.01	54	90	.00	131	250+	40	67
	131 (17)			61	32	52	61	4.5	135	337	.993	415	625	99.74	.06	55	90	.08	146	11	44	72
	132 (31)	Venezuela, Mississippi	V	60	19	32	250+	10.0	121	222	1.024	575	585	99.91	.00	55	92	.01	129	250+	39	65
	133 (70)			70	27	39	246	16.3	121	254	1.020	480	575	99.80	.03	60	86	.07	130	250+	46	66
	134 (82)			67	27	40	174	6.8	125	262	.995	540	630	99.82	.01	58	87	+.05	136	115	46	69
	134a (82)			59	25	42	196	5.5	128	292	996	540	635	99.85	+.01	54	92	+.04	138	81	43	73
	135 (109)			59	27	46	147	5.3	129	339	1.010	545	615	99.85	.00	53	90	.05	138	39	42	71
Region 3	136 (10)	Venezuela, Mexico	S	62	25	40	250+	13.2	126	365	1.038	450	515	99.83	.09	56	90	.44	132	137	38	61
	137 (11)	Texas	S, V	55	23	42	250	7.3	127	293	1.033	440	580	99.82	.10	48	87	14	136	143	35	64
	138 (33)	Venezuela	S, V	59	20	38	250+	9.5	124	272	1.024	495	580	99.77	.04	51	86	.10	132	250+	38	64
Region 4	139 (1)	do	V	63	21	33	200	14.5	125	321	1.035	495	515	99.84	.05	58	92	.20	134	201	40	63
	139a (1)	do	V	61	24	39	250+	10.0	126	327	1.035	500	530	99.81	.03	54	89	.06	135	182	39	64
	140 (7)	Texas	V	62	24	39	233	7.3	123	300	1.034	490	590	99.86	.03	55	89	.09	133	231	41	66
	140a (7)	do	V	61	20	33	250+	5.5	125	341	1.031	500	600	99.88	.01	58	95	.01	138	62	42	69
	141 (8)	Midcontinent	V, O	67	25	37	87	3.8	129	236	.996	515	640	99.83	.03	59	88	.02	137	44	46	69
	142 (13)			57	20	35	250+	5.3	125	247	1.004	580	699	99.78	.00	49	86	+.06	130	172	37	65
	143 (31)		V, S	62	23	37	250+	10.3	122	238	1.024	530	580	99.80	.00	54	87	+.09	129	250+	44	66
	144 (45)	Wyoming	V	62	20	32	203	6.8	123	205	1.026	490	630	99.76	.01	53	86	+.06	129	250+	39	63
	145 (46)	Arkansas	V, S	55	21	38	250+	4.5	126	293	1.019	585	665	99.79	.00	50	91	+.09	132	183	41	75
	146 (59)	Texas, Wyoming	S, O	60	23	38	133	6.5	123	195	1.029	500	580	99.70	.03	52	87	.06	133	188	38	63
	146a (59)	do	S, O	60	25	41	235	6.3	124	241	1.014	505	600	99.90	.06	57	93	.08	133	228	42	69
	146b (59)	do	S, O	57	22	39	198	5.5	125	209	1.023	505	585	99.83	.03	51	89	.04	133	165	38	64
	147 (67)	Texas	V, O	59	18	31	228	4.8	123	185	1.017	505	610	99.87	.04	52	88	.10	130	218	37	63
	148 (70)	Venezuela	S, V, B	58	25	43	218	11.8	126	277	1.020	510	585	99.81	.02	54	93	.04	133	172	41	71
	149 (73)	Wyoming, Oklahoma	V	66	24	36	225	7.0	119	190	1.016	475	625	99.86	.05	55	83	.05	129	250+	41	62
	150 (89)	Midcontinent	V, O	60	25	42	223	6.3	127	268	1.000	540	650	99.84	.01	55	92	+.05	133	208	44	73
	151 (93)	Wyoming	V	63	28	44	241	6.5	129	213	1.030	480	585	99.75	.04	49	78	.04	133	250+	35	56
	152 (94)	Midcontinent, Wyoming	V, S, O	64	17	27	218	2.8	121	178	1.014	565	630	99.82	.01	58	91	+.08	126	250+	45	70
	139x (1)	Venezuela		60	22	37	250+	13.0	126	351	1.038	500	525	99.92	.02	54	90	.08	136	237	38	63
	140x (7)	Texas		64	28	42	170	6.5	128	349	1.032	470	585	99.78	.04	57	89	.03	141	45	42	66
	144x (45)	Wyoming		65	21	32	250+	9.0	121	205	1.027	375	615	99.93	.13	54	83	.08	131	250+	38	58
	145x (46)	Arkansas		60	20	33	250+	5.0	123	293	1.025	555	675	99.94	.00	53	88	+.07	134	250+	41	68
	146x (59)	Texas		59	25	42	250+	7.0	122	251	1.013	480	585	99.79	.06	53	90	.00	131	230	41	69
	151x (93)	Wyoming		63	18	29	246	8.5	125	217	1.029	460	600	99.92	.05	52	83	.00	132	250+	36	57
Region 6	153 (7)	Texas	V	67	26	39	203	8.3	125	288	1.033	480	585	99.65	.04	59	88	.07	132	203	42	63
	154 (23)	do	V, S	63	16	25	212	6.0	122	144	1.033	530	600	99.73	.01	56	89	+.04	127	176	39	62
	155 (27)	Midcontinent	V, S, O	60	22	37	250+	5.8	124	253	1.005	535	630	99.80	.00	55	92	+.01	131	181	40	67
	156 (35)	Texas	V, S	67	27	40	170	6.5	125	343	1.030	550	615	99.56	.01	62	93	+.09	135	64	46	69
	157 (41)	do	P, O	61	18	30	250+	7.3	120	210	.999	575	650	99.79	+.01	57	93	+.09	124	245	46	75
	158 (46)	Wyoming	V, S, B	56	22	39	250+	5.8	126	304	1.020	580	670	99.87	.01	54	96	+.08	133	157	41	73
	159 (48)	Oklahoma	V, O	66	40	61	62	5.3	134	373	1.003	400	485	99.81	.15	62	94	.68	147	10	44	67
	160 (49)	Arkansas	V, S, B, O	59	28	47	237	5.3	128	356	1.017	560	630	99.88	.00	55	93	+.06	137	54	43	73
	161 (54)	Mississippi	S	62	34	55	142	5.5	131	379	1.034	495	525	99.94	.06	54	87	.25	144	37	40	65
	162 (57)	Gulf Coast, Midcontinent.	V, P, O	68	24	35	210	4.8	124	165	1.006	515	605	99.65	.04	63	93	.12	129	165	48	71
	163 (65)	Mississippi	S	62	26	42	172	5.8	129	366	1.032	475	520	99.72	.06	52	84	.17	142	65	39	63
	164 (87)	Gulf Coast	V	62	26	42	209	8.5	121	245	1.034	535	600	99.82	+.01	56	90	+.02	134	131	41	66
	165 (104)	Oklahoma	V, S	61	26	43	242	7.5	127	256	1.016	475	565	99.81	.00	52	85	.03	130	74	40	66
	166 (111)	Arkansas	V	53	24	45	204	7.5	127	337	1.027	540	630	99.67	.00	48	91	.01	134	181	38	72

[1] For States included in each of the Bureau of Public Roads regions, see inside front cover. No samples of 60–70 grade asphalts received from regions 5, 7, 8, and 9.
[2] Numbers in parentheses indicate refinery code number.
[3] V=vacuum distillation, S=steam distillation, O=blowing (oxidation), B=blending (different grades of asphalt), P=propane fractionation, and F=fluxing (heavy oils).
[4] Plus values represent gains rather than losses.

Table 2.—Test characteristics of 70–85 penetration grade asphalts

B.P.R. region[1]	Sample identification[2]	Source of crude	Method of refining[3]	Penetration			Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Soluble in CCl₄	Standard oven test at 325° F., 5 hours			Thin-film oven test, ⅛-inch film at 325° F., 5 hours				
																Penetration of residue			Tests on residue			
				100 g., 5 sec. at 77° F.	200 g., 60 sec. at 39.2° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute 77° F.	1 cm. per minute 39.2° F.				Pensky-Martens closed cup	Cleveland open cup		Loss	Value	Percent of original	Loss[4]	Softening point	Ductility, 5 cm. per min., 77° F.	Penetration at 77° F.	Percent of original penetration
							Cm.	*Cm.*	*°F.*	*Sec.*		*°F.*	*°F.*	*Pct.*	*Pct.*			*Pct.*	*°F.*	*Cm.*		
Region 2	167 (82)			67	27	40	174	6.8	125	240	0.995	540	630	99.82	0.01	58	87	+0.05	136	115	46	69
	168 (88)	Gulf Coast	V	71	31	44	250+	10.8	123	222	1.024	500	575	99.83	.03	64	90	.09	132	178	48	68
Region 4	169 (7)	Texas	V	66	26	39	243	8.3	125	266	1.038	475	570	99.87	.07	57	86	.21	133	250+	41	62
	170 (8)	Midcontinent	V, O	73	28	38	111	5.8	123	175	.996	540	625	99.83	.03	71	97	.03	130	58	55	75
	171 (15)	Wyoming	V	73	25	34	207	13.3	120	176	1.028	495	595	99.85	.03	65	89	.05	128	250+	45	62
	172 (19)	Louisiana, Wyoming	V	74	30	41	173	7.0	121	199	.995	430	690	99.92	.01	68	92	.06	125	170	59	80
	173 (27)	Midcontinent	S, V, O	82	27	33	174	9.3	118	210	1.023	420	650	99.80	.11	71	87	.17	129	202	49	60
	174 (43)	Wyoming	V	72	25	35	190	11.3	121	158	1.031	480	585	99.84	.03	62	86	+.02	128	186	45	63
	175 (47)	Canada	V	79	18	23	179	7.8	115	110	1.009	540	615	99.93	.00	67	85	+.07	125	177	47	60
	176 (59)	Wyoming, Texas	S, O	72	29	40	110	7.3	121	153	1.019	500	550	99.91	.03	65	90	.08	132	170	45	63
	177 (60)	Midcontinent	S, V, O	71	33	46	107	5.5	125	270	.999	495	625	99.74	.02	63	89	.01	137	33	48	68
	178 (64)	Wyoming, Texas	V, O	82	28	34	187	7.5	118	173	1.004	515	630	99.84	.01	71	87	+.04	127	189	54	66
	178a (64)	do	V, B, O	75	25	33	198	6.5	120	158	1.010	580	630	99.81	.01	67	89	+.08	126	222	51	68
	179 (67)	Texas	V, O	64	21	33	192	6.3	119	176	1.016	480	600	99.90	.04	56	88	.08	127	250+	39	61
	180 (74)	Wyoming, Oklahoma	V	66	24	36	225	7.0	119	245	1.016	475	625	99.86	.05	55	83	.05	129	250+	41	62
	181 (78)	Kansas	S, V, B	72	28	39	189	5.5	124	192	1.007	470	605	99.79	.04	63	88	.07	133	34	44	61
	182 (81)			72	34	47	211	6.5	125	206	1.012	545	610	99.78	.00	66	92	+.06	135	76	51	71
	183 (89)	Midcontinent	V, O	69	31	45	167	7.5	123	253	.994	540	630	99.88	.02	64	93	+.01	133	146	52	75
	184 (91)	Wyoming	V	75	28	37	250+	9.3	121	199	1.019	525	610	99.73	.01	63	84	.01	129	250+	47	63
	185 (93)	Midcontinent, Wyoming.	S, V, B	75	26	35	174	9.3	119	175	1.012	465	610	99.81	.07	69	92	.11	126	167	51	68
	186 (94)	do	S, V, B, O	63	19	30	250+	7.3	120	176	1.014	530	630	99.82	.00	51	81	+.09	128	207	43	68
	187 (96)	Wyoming	V, O	82	33	40	160	30.5	120	205	1.023	475	520	99.86	.07	73	89	.32	131	167	49	60
Region 5	188 (20)	Midcontinent	V	65	24	37	100	5.0	126	208	1.002	450	695	99.81	.00	59	91	.03	139	14	45	69
	189 (25)	Kansas	V	89	36	40	153	7.0	120	222	.999	435	665	99.70	.03	82	92	.01	131	62	59	66
	190 (55)	Kansas, Oklahoma, Texas.	V, P, B	70	26	37	206	8.3	119	233	1.004	565	660	99.87	.00	64	91	+.03	129	183	49	70
	191 (61)	Texas, Wyoming, Midcontinent.	V, O, P	66	29	44	218	7.5	123	192	1.029	505	550	99.81	.04	58	88	.13	133	162	41	62
	192 (68)	Texas	V, O	69	21	30	195	7.3	121	180	1.015	505	610	99.64	.04	62	90	.06	124	225	44	64
	193 (74)	Wyoming, Oklahoma	V	69	22	32	241	9.0	120	189	1.014	540	620	99.88	.01	62	90	+.03	128	228	44	64
	194 (87)	Texas	V, B, O	71	31	44	250+	8.3	121	228	1.015	505	590	99.89	.00	64	90	.00	132	160	47	66
	195 (111)	Arkansas	V, S	73	29	40	237	20.8	120	249	1.024	540	630	99.81	.02	66	90	+.02	127	250+	52	71

[1] For States included in each of the Bureau of Public Roads regions, see inside front cover. No samples of 70–85 grade asphalts received from regions 1, 3, 6, 7, 8, and 9.
[2] Number in parentheses indicate refinery code number.
[3] V=vacuum distillation, S=steam distillation, O=blowing (oxidation), B=blending (different grades of asphalt), P=propane fractionation, and F=fluxing (heavy oils).
[4] Plus values represent gains rather than losses.

Table 3.—Test characteristics of 120–150 penetration grade asphalts

B.P.R. region[1]	Sample identification[2]	Source of crude	Method of refining[3]	Penetration			Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Soluble in CCl₄	Standard oven test at 325° F., 5 hours			Thin-film oven test, ⅛-inch film at 325° F., 5 hours				
															Loss[4]	Penetration of residue		Loss[4]	Tests on residue			
				100 g., 5 sec. at 77° F.	200 g., 60 sec. at 39.2° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute at 77° F.	1 cm. per minute at 39.2° F.				Pensky-Martens closed cup	Cleveland open cup			Value	Percent of original		Softening point	Ductility, 5 cm per min., 77° F.	Penetration at 77° F.	Percent of original penetration
							Cm.	*Cm.*	*°F.*	*Sec.*		*°F.*	*°F.*	*Pct.*	*Pct.*			*Pct.*	*°F.*	*Cm.*		
Region 1	196 (22)	Mexico	S	135	61	45	184	250+	113	250	1.034	430	495	99.67	0.22	111	82	0.93	129	235	68	50
	197 (22)	do	S	131	47	36	229	96	110	246	1.029	430	490	99.72	.26	103	79	1.12	129	186	61	47
	198 (30)	Colombia		132	45	34	132	124	108	156	.999	500	635	99.74	.01	111	84	+.06	114	120	88	67
	199 (77)	Venezuela, Texas	V, S	128	45	33	141	182	110	152	1.018	405	530	99.68	.24	110	86	.73	119	229	72	56
Region 2	200 (1)			142	55	39	162	173	110	175	1.030	455	490	99.74	.10	120	85	.54	123	190	73	51
	201 (3)	Venezuela	V, S	127	46	36	182	250+	111	158	1.026	425	475	99.59	.21	105	83	1.10	123	192	62	49
	202 (9)	do	S	135	54	40	185	173	111	231	1.030	420	500	99.82	.27	112	83	1.18	123	213	64	47
	203 (14)			128	47	37	172	250+	108	125	1.020	495	550	99.71	.03	113	88	.10	117	227	79	62
	204 (17)			126	51	40	145	15.5	112	163	.989	410	640	99.62	.07	118	94	.02	123	114	75	60
	205 (31)	Venezuela		135	53	39	190	135	109	146	1.014	480	555	99.75	.02	115	85	.02	119	210	82	61
	206 (70)			121	50	41	218	232	109	160	1.015	495	575	99.73	.03	107	88	.04	119	250+	76	63
Region 4	207 (1)	Venezuela	V, S	138	50	36	200	173	111	188	1.028	470	485	99.73	.10	130	94	.58	122	240	74	54
	207a (1)	do	V, S	132	47	36	147	154	111	190	1.029	475	500	99.72	.09	115	87	.53	122	217	72	55
	208 (8)	Midcontinent	V, O	152	40	26	87	10	108	105	.992	510	655	99.75	.04	143	94	.04	115	126	91	60
	209 (13)	Illinois	S, V	130	38	29	141	41	110	128	1.003	465	620	99.70	.04	107	82	.03	120	160	72	55
	210 (51)	Wyoming	V	128	44	34	138	250+	110	111	1.026	485	555	99.78	.04	111	87	.06	118	185	74	58
	211 (59)	Wyoming, Texas	S, O	151	49	34	97	56.5	108	93	1.017	500	550	99.58	.03	126	83	.05	116	133	86	57
	211a (59)	do	S, O	121	42	35	141	67.5	111	122	1.018	500	560	99.89	.02	104	87	.04	119	191	72	60
	212 (60)	Midcontinent	S, V, O	142	48	34	110	12.5	109	136	.995	550	640	99.61	.01	118	83	+.07	119	164	84	59
	213 (64)	Wyoming, Texas	V	136	36	26	148	48.5	108	106	1.010	495	610	99.77	.13	110	81	.15	117	162	73	54
	213a (64)	do	V	138	35	25	125	58.5	107	110	1.011	475	630	99.78	.01	112	81	+.07	116	180	77	56
	214 (67)	Texas	V, O	138	36	26	124	45	113	107	1.010	440	605	99.73	.09	115	83	.10	117	163	76	55
	215 (89)	Midcontinent	V, O	117	41	35	128	75.5	113	135	.993	545	630	99.83	.02	108	92	+.02	122	125	83	71
	216 (94)	Midcontinent, Wyoming	S, V, O	143	40	28	115	31	109	121	.999	470	625	99.76	.05	124	87	.01	116	146	87	61
	216a (94)	do	S, V, O	138	49	36	85	13.8	110	124	.993	540	625	99.71	.01	123	89	.06	117	98	93	67
	217 (96)	Wyoming	V, O	133	46	35	118	84	111	146	1.021	455	520	99.67	.09	114	86	.27	119	145	76	57
Region 5	218 (38)	do	V	135	45	33	141	117	110	113	1.021	390	565	99.68	.12	108	80	.17	119	229	71	53
	219 (43)	do	V	152	53	35	155	132	106	97	1.026	380	555	99.90	.11	119	78	.24	117	240	77	51
	220 (51)	do	S, V	125	44	35	155	182	110	123	1.025	500	580	99.69	.03	107	86	.05	120	234	71	57
	221 (52)	do	S, F	120	50	42	144	250+	110	128	1.028	360	665	99.77	.19	102	85	.27	121	204	67	56
	222 (79)	do	V, S, B	130	33	25	141	22	110	105	1.019	425	615	99.68	.07	108	83	.06	117	173	73	56
	223 (96)	do	V, O	126	48	38	125	250+	111	145	1.022	470	535	99.61	.07	110	87	.23	121	150	74	59
Region 6	224 (7)	Texas	V	121	44	36	170	135	112	187	1.024	500	580	99.72	.00	112	93	.02	122	182	73	60
	225 (21)	do	V, O, B, F	132	41	31	122	11.8	109	109	1.023	485	580	99.71	.04	110	83	.08	120	115	68	52
	226 (23)	do	S, V	126	37	29	118	68.5	109	86	1.023	500	570	98.82	.02	112	89	.01	118	120	72	57
	227 (27)	Midcontinent	S, V	127	37	29	150	14.3	109	150	1.000	445	655	99.61	.05	102	80	.09	118	154	77	61
	228 (35)	Texas	S, V	140	58	41	154	26.5	113	156	1.021	520	585	99.76	.00	112	80	+.04	124	109	83	59
	229 (41)	do	P, O	130	29	22	126	94	109	167	.988	535	645	99.75	.00	114	88	+.06	113	145	90	69
	230 (46)	Arkansas	S, V	122	40	33	173	17.8	111	166	1.014	585	655	99.60	+.01	112	92	+.09	117	215	80	66
	231 (48)	Oklahoma	V, O	141	68	48	195	20.3	113	165	1.001	430	485	99.74	.17	123	87	.74	127	91	78	55
	232 (49)	Arkansas	S, V	140	48	34	142	250+	109	174	1.013	580	630	99.64	+.01	125	89	+.12	117	175	92	66
	233 (54)	Mississippi	S	125	67	54	177	20.5	116	180	1.026	415	500	99.82	.16	96	77	.63	133	90	62	50
	234 (57)	Gulf, Midcontinent	V, B, O	140	48	34	109	9.3	109	95	.995	485	585	99.95	.07	122	87	.15	122	93	84	60
	235 (65)	Mississippi	S, B	127	61	48	166	34.5	114	186	1.025	410	510	99.74	.11	106	83	.41	127	111	70	55
	236 (72)	Texas, Kansas	V, P, B	125	33	26	146	6.3	114	213	.981	590	655	99.66	+.01	119	95	+.04	119	87	83	64
	237 (85)	Gulf, Mexico	V	121	40	33	135	250+	111	94	1.023	500	525	99.59	.01	118	98	.13	114	130	84	69
	238 (87)	Gulf Coast	V	128	44	34	157	144	110	169	1.020	540	590	99.76	.00	112	88	.04	119	250+	83	65
	239 (92)	Texas	V, O	129	47	37	142	250+	106	72	1.009	455	515	99.69	.07	119	92	.37	111	217	100	78
	240 (95)	do	S, V, O	137	52	38	135	40	109	120	.998	475	600	99.73	.04	114	83	.03	119	164	80	58
	241 (97)	Oklahoma	V	130	38	29	110	133	112	166	1.017	470	690	99.75	.01	116	89	.07	115	165	83	64
	242 (104)	do	S, V	120	44	37	158	19.8	111	136	1.010	485	560	99.70	.01	102	85	.01	122	195	71	59
	243 (106)	do	V, B	134	44	33	205	12.3	112	157	.990	390	660	99.83	.06	112	84	.05	121	162	76	57
	244 (111)	Arkansas	V	127	51	40	170	99	110	181	1.020	550	650	99.78	.01	118	93	.00	116	169	87	69
	245 (111)	do	V	127	47	37	138	23.5	111	141	1.019	535	615	99.71	.02	112	88	+.05	120	183	77	61
	246 (24)	California	S, V	130	52	40	175	82	111	145	1.024	430	470	99.74	.18	120	92	.98	123	250+	70	54
	247 (40)	do	V, B, O	136	42	31	135	250+	104	71	1.005	470	525	99.71	.06	124	91	.14	111	137	93	68
	248 (58)	do	S, V	122	41	34	158	232	110	103	1.015	460	510	99.84	.10	104	85	.11	117	159	70	57
	249 (66)	do	S, V, B	124	44	33	193	250	110	72	1.012	500	545	99.84	.04	110	89	.20	113	200	79	64
	250 (99)	do	S, F	121	44	36	166	77	112	160	1.027	440	480	99.88	.14	100	83	1.02	130	115	57	47
	251 (112)	do	S, V	141	50	35	188	157	109	155	1.029	420	470	99.85	.22	109	77	1.43	129	175	59	42

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[1] For States included in each of the Bureau of Public Roads regions, see inside front cover. No samples of 120–150 grade asphalts received from regions 3, 7, 8, and 9.

[2] Numbers in parentheses indicate code designation of refinery.

[3] V=vacuum distillation, S=steam distillation, O=blowing (oxidation), B=blending (different grades of asphalt), P=propane fractionation, and F=fluxing (heavy oils).

[4] Plus values represent gains rather than losses.

Table 4.—Test characteristics of 150–200 penetration grade asphalts

B.P.R. region[1]	Sample identification[2]	Source of crude	Method of refining[3]	Penetration			Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Soluble in CCl4	Standard oven test at 325° F., 5 hours			Thin-film oven test, ⅛-inch film at 325° F., 5 hours				
				100 g., 5 sec. at 77° F.	200 g., 60 sec. at 39.2° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute at 77° F.	1 cm. per minute at 39.2° F.				Pensky-Martens closed cup	Cleveland open cup		Loss[4]	Penetration of residue		Loss[4]	Tests on residue			
																Value	Percent of original		Softening point	Ductility, 5 cm. per min., 77° F.	Penetration at 77° F.	Percent of original penetration
							Cm.	*Cm.*	*° F.*	*Sec.*		*° F.*	*° F.*	*Pct.*	*Pct.*			*Pct.*	*° F.*	*Cm.*		
Region 3	252 (6)	Venezuela, Mississippi	S, V, B	153	58	38	167	200+	107	164	1.028	440	475	99.71	0.15	135	88	0.76	122	156	78	50
	253 (10)	Venezuela, Mexico	S	189	81	43	147	200+	107	173	1.031	415	460	99.65	.33	147	78	1.48	125	178	78	41
	254 (11)	Texas	S, F	175	68	39	133	200+	106	136	1.022	355	555	99.64	.24	141	81	.57	117	225	85	48
	255 (33)	Venezuela	S, V	181	70	39	119	200+	104	122	1.015	495	545	99.67	.04	154	85	.12	115	175	101	56
	256 (42)	Mississippi	S, V	182	68	37	105	200+	106	135	1.026	435	515	99.73	.08	147	81	.30	115	156	100	55
	257 (63)	do	S	158	85	54	142	134	109	149	1.024	415	480	99.74	.20	132	84	.86	127	140	76	48
	258 (69)	Venezuela	B	173	65	38	160	200+	103	114	1.015	505	555	99.71	.02	152	88	.02	111	218	109	63
	259 (83)	Mississippi	S, V	165	71	43	133	102	107	146	1.029	460	505	99.72	.15	154	93	.29	120	185	91	55
	259a (83)	do	S, V	161	66	41	148	35	107	119	1.002	480	595	99.85	.03	137	85	.04	120	129	96	60
	260 (102)	do	S	166	87	52	144	127	108	148	1.023	395	475	99.74	.30	131	79	1.16	128	154	73	42
	261 (103)	do	S	162	85	52	120	15	109	252	1.021	425	490	99.91	.23	137	85	.98	127	158	79	49
Region 5	262 (18)	Wyoming	V	193	65	34	112	200+	101	79.4	1.022	325	610	99.73	.45	137	71	.60	115	165	83	43
Region 8	263 (16)			151	30	20	146		104		1.068	420	500	99.47	.22	124	82	.73	116	200	65	43
	264 (18)	Wyoming	V	164	43	26	150	40	106	83.0	1.022	590	625	99.72	+.03	134	82	+.12	113	123	94	57
	265 (26)			151	57	38	94	44	110	114	1.024	390	470	99.81	.37	123	81	1.17	127	77	65	43
	266 (38)			176	54	31	89	200+	102	95.2	1.028	465	560	99.81	.06	147	84	.04	115	178	87	49

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[1] For States included in each of the Bureau of Public Roads regions, see inside front cover. No samples of 150–200 grade asphalts received from regions 1, 2, 4, 6, 7, and 9.

[2] Numbers in parentheses indicate code designation refinery.

[3] V=vacuum distillation, S=steam distillation, O=blowing (oxidation), B=blending (different grades of asphalt), P=propane fractionation, and F=fluxing (heavy oils).

[4] Plus values represent gains rather than losses.

Table 5.—Test characteristics of asphalts of different grades from same refinery source

Sample identification[1]	Grade	Source of crude	Method of refining[2]	Penetration		Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Thin-film oven test, ⅛-inch film at 325° F., 5 hours			
				100 g., 5 sec. at 77° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute at 77° F.	1 cm. per minute at 39.2° F.				Pensky-Martens closed cup	Cleveland open cup	Loss[3]	Tests on residue		
														Softening point	Ductility, 5 cm. per min., at 77° F.	Percent of original penetration
						Cm.	Cm.	° F.	Sec.		° F.	° F.	Pct.	° F.	Cm.	
127 (1)	60-70	Venezuela	V, S	64	36	250+	9.8	122	358	1.035	495	540	0.05	133	183	66
13	85-100		V, S	89	36	175	18.3	120	255	1.033	480	530	.18	132	175	62
200	120-150		V, S	142	39	162	173	110	175	1.030	455	490	.54	123	190	51
128 (3)	60-70	Venezuela	V, S	60	42	250+	8.5	125	346	1.033	475	515	.36	139	140	62
14	85-100		V, S	92	35	185	20.5	117	232	1.030	440	490	.70	130	182	57
201	120-150		V, S	127	36	182	250+	111	158	1.026	425	475	1.10	123	192	49
153 (7)	60-70	Texas	V	67	39	203	8.3	125	288	1.033	480	585	.07	132	203	63
169	70-85		V	66	39	243	8.3	125	266	1.038	475	570	.21	133	250+	62
67	85-100		V	90	38	201	20.8	122	243	1.029	460	575	.08	129	192	61
224	120-150		V	121	36	170	135	112	187	1.024	500	580	.02	122	182	60
141 (8)	60-70	Midcontinent	V, O	67	37	87	3.8	129	236	.996	515	640	.02	137	44	69
170	70-85		V, O	73	38	111	5.8	123	175	.996	540	625	.03	130	58	75
38	85-100		V, O	90	36	170	6.3	120	189	1.000	510	605	.00	129	120	62
208	120-150		V, O	152	26	87	10.0	108	105	.992	510	655	.04	115	126	60
129 (9)	60-70	Venezuela	S	64	36	215	24.0	126	347	1.038	455	505	.46	137	113	59
15	85-100		S	96	35	189	33.5	118	249	1.036	420	490	.64	132	176	55
202	120-150		S	135	40	185	173	111	231	1.030	420	500	1.18	123	213	47
130 (14)	60-70			60	33	250+	10.5	122	213	1.024	500	575	.00	131	250+	67
17	85-100			93	37	196	63	116	166	1.022	535	565	+.04	123	205	66
203	120-150			128	37	172	250+	108	125	1.020	495	550	.10	117	227	62
131 (17)	60-70			61	52	61	4.5	135	337	.993	415	625	.08	146	11	72
18	85-100			87	46	116	7.3	124	239	.992	425	630	.08	135	29	66
204	120-150			126	40	145	15.5	112	163	.989	410	640	.02	123	114	60
121 (22)	60-70	Mexico	S	57	44	250+	11.3	128	431	1.039	480	545	.29	140	100	65
3	85-100		S	87	45	248	51.5	121	318	1.037	445	560	.55	134	138	60
196	120-150		S	135	45	184	250+	113	250	1.034	430	495	.93	129	235	50
154 (23)	60-70	Texas	V, S	63	25	212	6.0	122	144	1.033	530	600	.04	127	176	62
69	85-100		V, S	87	23	112	12.5	116	120	1.029	535	585	+.02	125	178	61
226	120-150		V, S	126	29	118	68.5	109	86	1.023	500	570	.01	118	120	57
155 (27)	60-70	Midcontinent	V, S, O	60	37	250+	5.8	124	253	1.005	535	630	+.01	131	181	67
70	85-100		V, S	87	32	150	6.8	120	208	1.005	570	635	+.03	127	250+	61
227	120-150		V, S	127	29	150	14.3	109	150	1.000	445	655	.09	118	154	61
122 (30)	60-70	Colombia		64	34	187	10.0	121	247	1.011	500	640	+.07	129	250+	73
6	85-100			86	35	180	30.5	115	200	1.005	525	580	+.10	122	250+	71
198	150-200			132	34	132	124	108	156	.999	500	635	+.06	114	120	67
138 (33)	60-70	Venezuela	V, S	59	38	250+	9.5	124	272	1.024	495	580	.10	132	250+	64
27	85-100		V, S	93	37	195	88	113	153	1.025	495	545	.13	123	221	66
255	150-200		V, S	181	39	119	200+	104	122	1.015	495	545	.12	115	175	56
156 (35)	60-70	Texas	V, S	67	40	170	6.5	125	343	1.030	550	615	+.09	135	64	69
73	85-100		V, S	93	35	234	10.3	116	239	1.025	465	625	.05	125	209	64
228	120-150		V, S	140	41	154	26.5	113	156	1.021	520	585	+.04	124	109	59
157 (41)	60-70	do.	P, O	61	30	250+	7.3	120	210	.999	575	650	+.09	124	245	75
74	85-100		V, P, O	94	23	250+	4.3	117	263	1.005	525	640	+.03	127	181	68
229	120-150		P, O	130	22	126	94	109	167	.998	535	645	+.06	113	145	69
159 (48)	60-70	Oklahoma	V, O	66	61	62	5.3	134	373	1.003	400	485	.68	147	10	67
76	85-100		V, O	93	52	107	7.5	125	257	1.002	425	475	.70	138	20	61
231	120-150		V, O	141	48	195	20.3	113	165	1.001	430	485	.74	127	91	55
160 (49)	60-70	Arkansas	V, S, B, O	59	47	237	5.3	128	356	1.017	560	630	+.06	137	54	73
77	85-100		V, S, B, O	94	37	213	9.8	117	238	1.016	550	635	.06	124	185	69
232	120-150		V, S	140	34	142	250+	109	174	1.013	580	630	+.12	117	175	66
161 (54)	60-70	Mississippi	S	62	55	142	5.5	131	379	1.034	495	525	.25	144	37	65
78	85-100		S	92	52	139	9.3	122	240	1.030	400	490	.42	138	36	58
233	120-150		S	125	54	177	20.5	116	180	1.026	415	500	.63	133	90	50
162 (57)	60-70	Gulf Coast, Midcontinent.	V, P, O	68	35	210	4.8	124	165	1.066	515	605	.12	129	165	71
79	85-100		V, P, O, B	90	26	145	6.0	118	123	1.001	485	565	.20	127	114	70
234	120-150		V, O, B	140	34	109	9.3	109	95	.995	485	585	.15	122	93	60
146 (59)	60-70	Texas, Wyoming	S, O	60	38	133	6.5	123	195	1.029	500	580	.06	133	188	63
47a	85-100		S, O	88	32	202	24.0	118	160	1.021	515	575	.03	126	190	59
211a	120-150		S, O	121	35	141	67.5	111	122	1.018	500	560	.04	119	191	60
163 (65)	60-70	Mississippi	S	62	42	172	5.8	129	366	1.032	475	520	.17	142	65	63
80	85-100		S	90	52	180	17.0	114	246	1.030	445	520	.20	128	102	63
235	120-150		S, B	127	48	166	34.5	114	186	1.025	410	510	.41	127	111	55
147 (67)	60-70	Texas	V, O	59	31	228	4.8	123	185	1.017	505	610	.10	130	248	63
179	70-85		V, O	64	33	192	6.3	119	176	1.016	480	600	.08	127	250+	61
50	85-100		V, O, B	98	30	140	8.0	114	135	1.015	435	610	.09	125	166	52
214	120-150		V, O	138	26	124	45.0	113	107	1.010	440	605	.10	117	163	55
124 (77)	60-70	Venezuela and Texas	S, V, O	64	39	228	8.0	122	295	1.021	435	580	.45	135	162	58
9	85-100		S, V, O	93	38	150	22.3	116	198	1.022	415	540	.50	130	226	57
199	120-150		S, V	128	33	141	182	110	152	1.018	405	530	.73	119	229	56
164 (87)	60-70	Gulf Coast	V	62	42	209	8.5	121	245	1.034	535	600	+.02	134	131	66
23	85-100		V	81	46	190	11.3	119	198	1.016	530	585	.04	127	208	70
238	120-150		V	128	34	157	144	110	169	1.020	540	590	.04	119	250+	65
150 (89)	60-70	Midcontinent	V, O	60	42	223	6.3	127	268	1.000	540	650	+.05	133	208	73
183	70-85		V, O	69	45	167	7.5	123	253	.994	540	630	+.01	133	146	75
53	85-100		V, O	86	40	151	7.3	119	195	.995	545	635	+.03	126	150	69
215	120-150		V, O	117	35	128	75.5	113	135	.993	545	630	+.02	122	125	71
165 (104)	60-70	Oklahoma	V, S	61	43	242	7.5	127	256	1.016	475	565	.03	130	74	66
87	85-100		V, S	88	39	166	12.3	118	180	1.012	515	570	+.05	128	162	63
242	120-150		V, S	120	37	158	19.8	111	136	1.010	485	560	.01	122	195	59
166 (111)	60-70	Arkansas	V, S	53	45	204	7.5	127	337	1.027	540	630	.01	134	181	72
195	70-85		V, S	73	40	237	20.8	120	249	1.024	540	630	+.02	127	250+	71
89	85-100		V, S	89	34	138	11.0	116	236	1.021	560	660	.09	124	160	72
244	120-150		V, S	127	40	170	99.0	110	181	1.020	550	650	.00	116	169	69

[1] Numbers in parentheses indicate refinery code number.

[2] V=vacuum distillation, S=steam distillation, O=blowing (oxidation), B=blending (different grades of asphalt), P=p
pane fractionation, and F=fluxing (heavy oils).

[3] Plus values represent gains rather than losses.

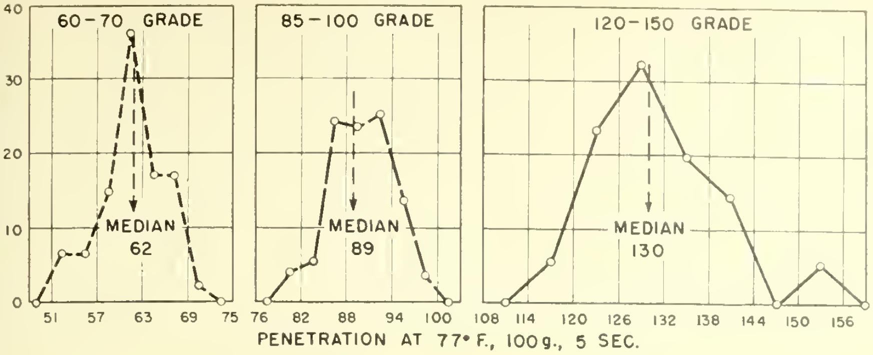


Figure 1.—Distribution of penetration.

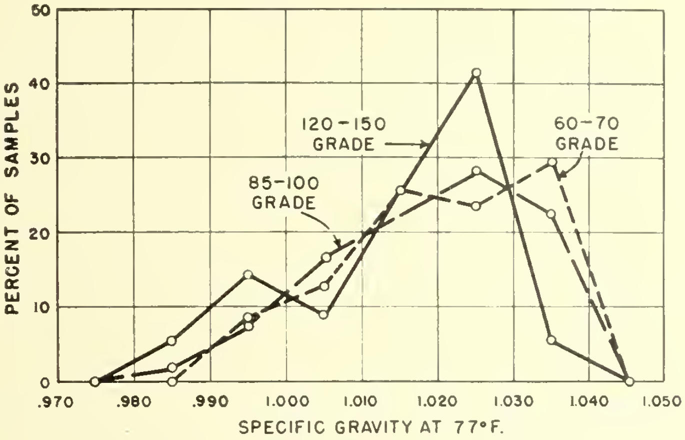


Figure 2.—Distribution of specific gravity.

ource or combination of sources by a specific finery and are assumed to represent con-urrent production.

Results of Tests

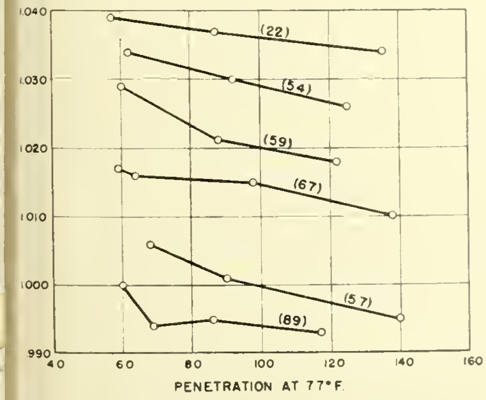
The following discussion of the frequency stribution graphs for the 60–70, 85–100, and 20–150 penetration grades points out the nge and variations in some of the test aracteristics. The relation of the penetra- on grades to the changes in some of the test aracteristics are also discussed.

enetration at 77° F.

Figure 1 shows the range and distribution the results of the penetration test at 77° F. r the three grades. Thirteen, or approxi- ately 28 percent, of the 47 samples of the 60–70 asphalts had penetration values of less than 60; none exceeded the upper limit of 70. In part 1, 12 samples, or approximately 10 percent, of the 85–100 grade had values below 85; and, there also, none exceeded the upper limit of 100. Of the 56 asphalts of the 120–150 grade, 1 sample was less than 120 and 3 exceeded 150, which constituted about 7 percent of the samples. There is a general trend for the median value for all of these grades to approach the lower limit of the specification rather than the upper. The medians were 62, 89, and 130 for the respective grades.

Specific gravity at 77° F.

The specific gravity of asphalt cements is seldom used as a specification requirement, but it is often needed to establish design criteria for bituminous mixtures. It may also be used to control the uniformity of the supply of asphalt from a refinery for a specific project. The general distribution data plotted in figure 2, indicate that the differences in the overall range of specific gravities for the three grades were not significant. The effect of penetration on specific gravity is better illustrated in figure 3 where some of the data given in table 5 are plotted to show the relation between these characteristics for the various grades of asphalt from the same sources. These graphs also show that for materials from the same source the specific



igure 3.—Relation of penetration at 77° F. to specific gravity at 77° F.

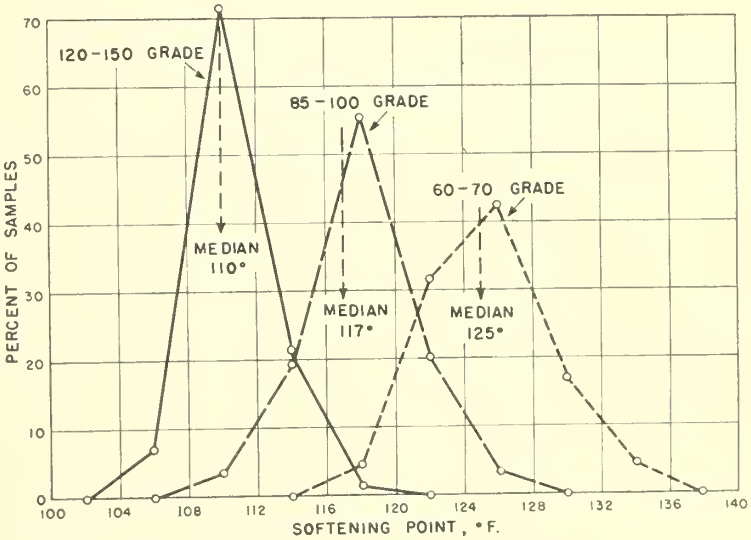


Figure 4.—Distribution of softening points.

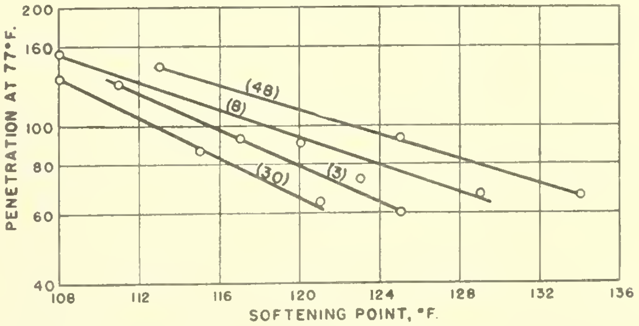


Figure 5.—Relation of penetration at 77° F. to softening point.

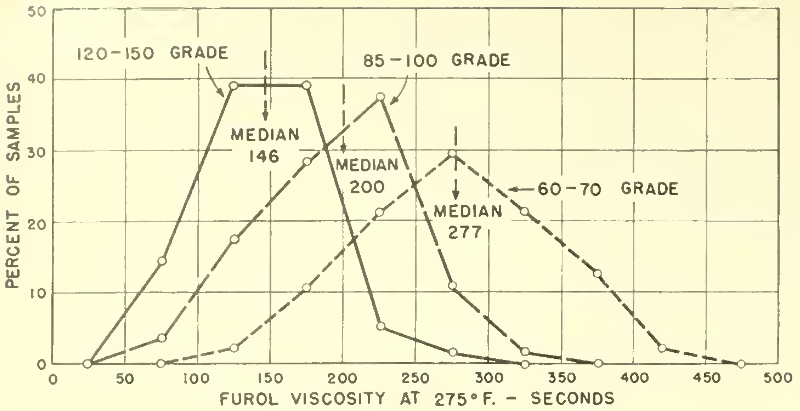


Figure 6.—Distribution of Furol viscosity.

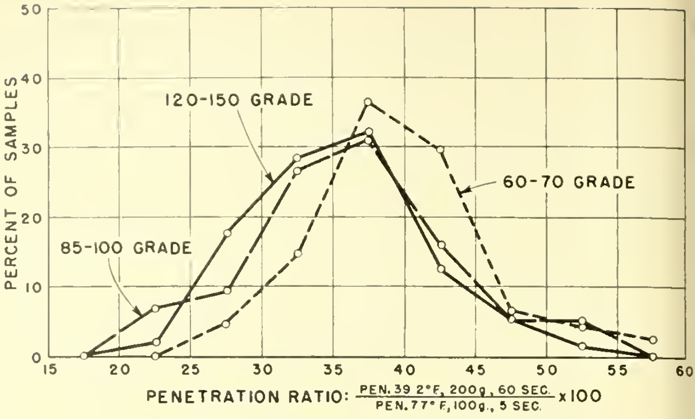


Figure 8.—Distribution of penetration ratios.

gravity decreased slightly as the penetration increased. (*Note:* The numbers in parentheses shown in this figure and subsequent figures illustrating the relation of penetration to other properties are the refinery code numbers.)

Softening point

Figure 4 shows that there was considerable overlapping of values of softening point for the asphalts of the 60–70, 85–100, and 120–150 grades but there was a definite difference in the point at which the peak occurred for each grade. The values of the medians were 125°, 117°, and 110° F. for the three grades. Data selected from table 5 and plotted in figure 5 illustrate the relation between softening point and penetration for different grades from the same refinery. It will be noted that the differences between values for different grades from the same source are essentially the same as the differences in the median values for all asphalts in each grade.

Furol viscosity at 275° F.

There was considerable overlapping of values for the Furol viscosity of the three penetration grades, as shown in figure 6. However, there was a distinct difference in the median values for the three grades—being 277 seconds, 200 seconds, and 146 seconds for the 60–70, 85–100, and 120–150 grades. The ranges in viscosity values for the three penetration grades were 144 to 431, 85 to 318, and 71 to 250 seconds, respectively.

The relation of penetration at 77° F. to Furol viscosity at 275° F. for asphalts of the different penetration grades from the same sources are illustrated in figure 7. The upper and lower lines represent the extreme viscosities for the asphalts. The upper line shows the penetration-viscosity relation for the three grades of asphalt refined from a Mexican crude by refinery 22. The asphalts from refinery 23 had the lowest viscosity for equivalent penetration of those represented by three or more grades given in table 5 but there were a few asphalts from other sources that had lower viscosities. The two asphalts refined from a California crude by refinery 66 had the lowest Furol viscosity for equivalent penetration for all the materials tested and therefore they were used to establish the lower viscosity line. Other penetration-viscosity relations shown are believed generally typical of materials from other sources.

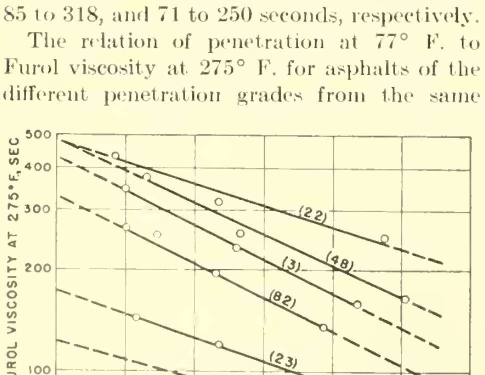


Figure 7.—Relation of penetration at 77° F. to Furol viscosity at 275° F.

The data selected for presentation in figure 7 generally fall into a straight line, but it should be noted that some of the asphalts of different grades from the same refinery did not show this straight-line relation. It was also not possible to determine from the data the extent to which the linear relation would be applicable to softer or harder materials than those tested. However, the data serve to show the wide difference in the high temperature viscosities of asphalts of the same grade. The significance of these differences are discussed later.

Penetration ratio

Values for the ratio of the penetration at 39.2° F., 200 g., 60 sec., to the penetration at 77° F., 100 g., 5 sec., as shown in figure 8, did not vary significantly for the 85–100 and 120–150 grades. The minimum and maximum values and the relative position of the distribution curve for the 60–70 asphalts indicated slightly higher penetration ratios than for the 85–100 and 120–150 asphalts.

Figure 9 illustrates the relation of the penetration at 77° F. to the penetration ratio for asphalts of different grades from the same refinery source. Some series of asphalts from the same source showed little change in penetration ratio for different grades. Others showed a decrease in penetration ratio with increase in penetration, while still other results were quite erratic and no uniform trend could be determined. In some cases this could have been attributed to the greater amount of blowing to reduce the lower penetration materials to grade.

A relation between the Furol viscosity at 275° F. and the penetration ratio might be expected since both values are assumed to be dependent on the viscosity-temperature relationship of the asphalt. However, when asphalts of different grades and sources were compared, no definite trend was established. This could have been attributable to differences in the plastic flow properties of various materials, or possibly due to the empirica nature of the penetration ratio.

Ductility at 77° F., 5 cm. per minute

Because of the influence of consistency, it i difficult to evaluate trends in the ductilit results for asphalts from different penetratio grades. Figure 10 shows that 38 percent o the 60–70 asphalts and 6 percent of the 85-100 samples had ductility values greate than 250 cm., the capacity of the ductilit machine. All of the 120–150 grade material had values less than 250 cm. Even thoug more 60–70 asphalts exceeded 250 cm. tha the other grades, there were also a greate number of samples of this grade that ha ductility values less than 100 cm., the minimum requirement most commonly foun in specifications.

Ductility at 39.2° F.

The frequency distribution for the ductili at 39.2° F., 1 cm. per minute is shown figure 11. While there was similarity in t polygons for the 60–70 and 85–100 grade the polygon for the 120–150 grade w radically different. The ductility rang were 2.8 to 24.0 cm. for the 60–70 grade,

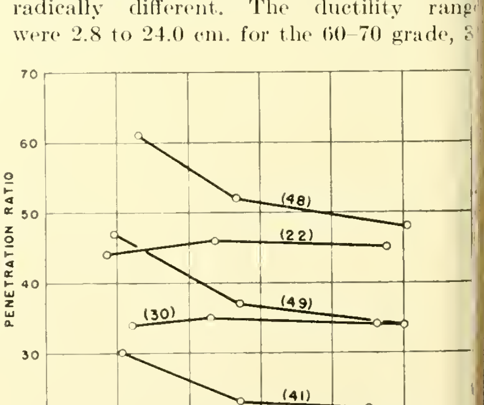


Figure 9.—Relation of penetration at 77° to penetration ratio.

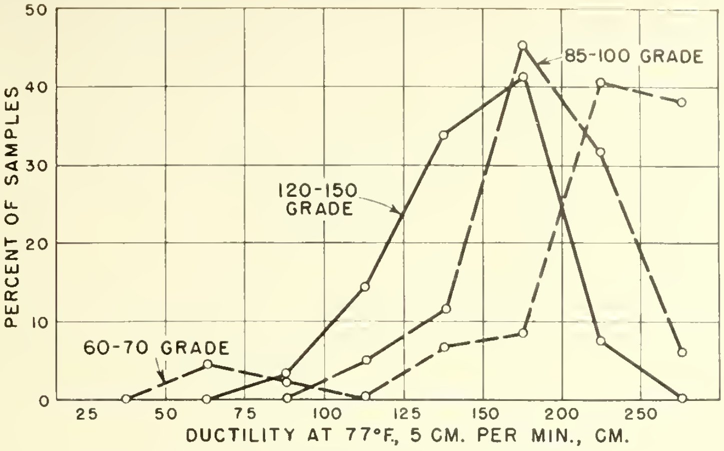


Figure 10.—Distribution of ductility.

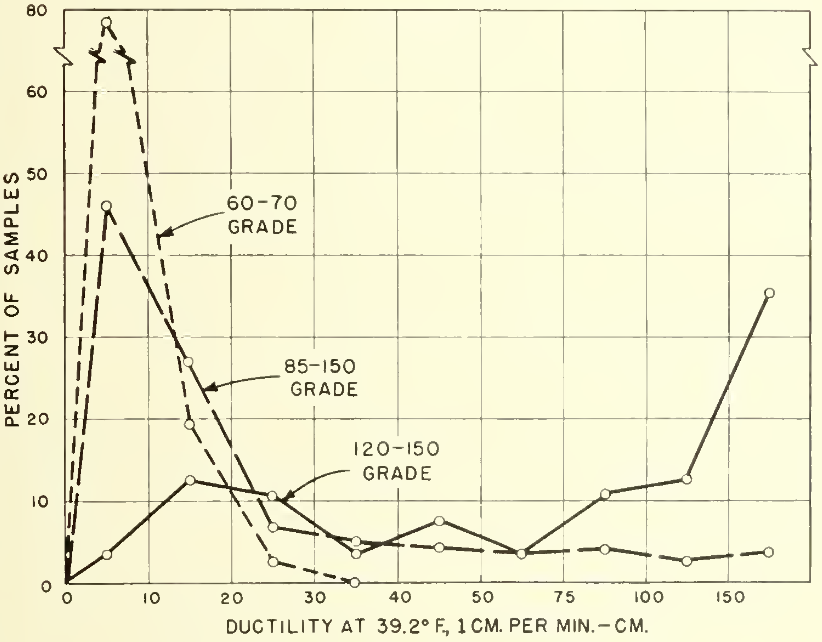


Figure 11.—Distribution of ductility at 39.2° F.

o 250+ cm. for 85–100 grade, and 6.3 to 50+ cm. for the 120–150 grade. The ercentage of samples that had ductilities ss than 10 cm. were 69 for the 60–70, 47 or the 85–100, and 4 for the 120–150 sphalts.

While it has been established that the uctilities of asphalts from different sources nd methods of refining vary greatly, it also as been established that consistency as easured by penetration has a pronounced ffect on ductility values at low temperatures. he influence of penetration on ductility at 9.2° F. and variability in ductility of ashalts from different sources is illustrated in gure 12. The wide difference in the ductiliies of asphalts from various sources when ompared on an equipenetration basis as well s the large decrease in ductility with derease in penetration for materials from the ame source are shown.

lash points

Figures 13 and 14 respectively, show the istribution of flash points determined by the ensky-Martens closed cup and the Cleveland pen cup apparatus. Very little difference was apparent between the results of flash points of the 85–100 and 120–150 grades, but the shift toward higher values for the 60–70 grade was appreciable. The relation between the results for the closed cup and open cup apparatus showed the same erratic pattern for the 60–70 and 120–150 grades as was noted for the 85–100 penetration asphalts. Also, there were no consistent trends indicated from the data in table 5 for asphalts of different grades from the same source. In a few samples there was a decrease in flash point with increase in penetration but in other samples this result was not evident.

Thin-film oven test, loss in weight

The change in weight during the heating of the asphalts in the ⅛-inch film for 5 hours at 325° F. is shown in figure 15. The positive values on the horizontal axis indicate a gain in weight and the negative values indicate a loss. Eighty-five percent of the 60–70 asphalts, 65 percent of the 85–100, and 66 percent of the 120–150 grade materials had slight gains or losses in weight less than 0.20 percent. Although not shown, the median values for the three grades were approximately the same. The most significant fact shown is that there were some asphalts of all three grades that had relatively high loss in weight.

Two percent of the 60–70 grade samples, 13 percent of the 85–100 grade samples, and 21 percent of the 120–150 grade samples had losses greater than 0.50 percent. However, where series of asphalts of different penetration grades from the various sources were compared the trend for higher losses in higher penetration asphalts was not consistent. This is evident in figure 16 where the change in weight is plotted against penetration for the different grades from the same source. It is of interest to note the two distinctive patterns of behavior. Generally, the materials that had significant losses showed a very definite trend toward higher loss for the increasing penetration, the relation being essentially linear. However, where a refinery produced asphalts with very little losses or with gains in weights, variation with grade was erratic and of little significance. The major exception to this trend was found in the analysis of the asphalts from refinery 48. The 60–70 grade asphalt from this refinery had a fairly high loss and the loss increased only slightly for the 85–100 and 120–150 penetration grades.

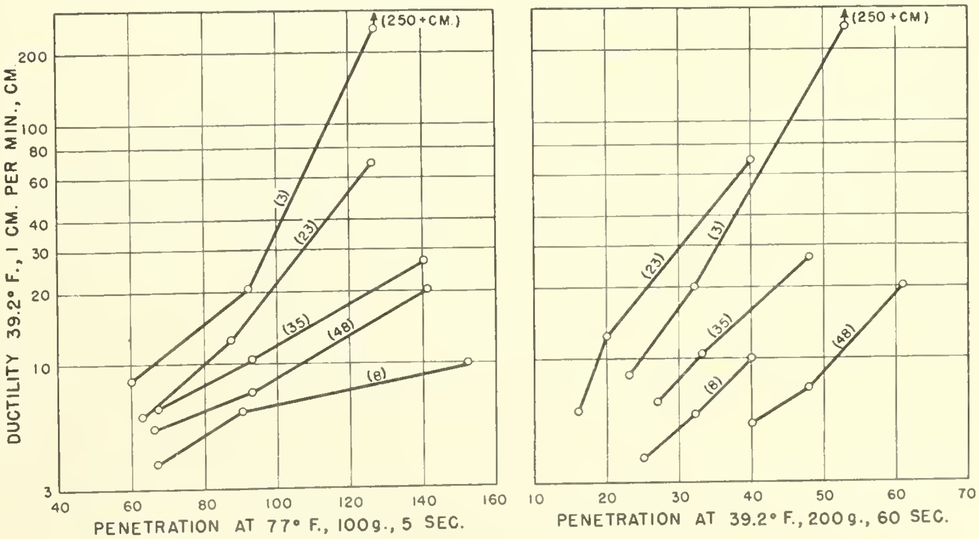


Figure 12.—Relation of penetration at 77° F. and 39.2° F. to ductility at 39.2° F.

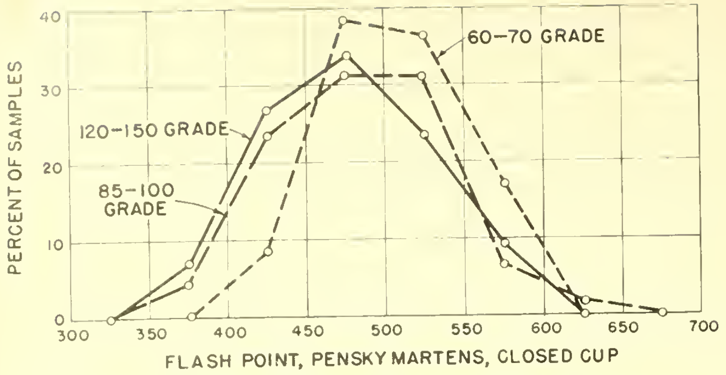


Figure 13.—Distribution of flash points (closed cup).

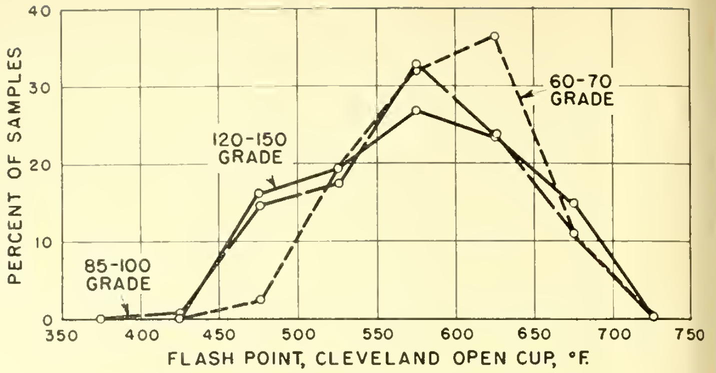


Figure 14.—Distribution of flash points (open cup).

Retained penetration, thin-film oven test

In conjunction with the comparisons of loss in weight in the thin-film tests, there also was a significant difference in the percentage of the original penetration retained by the residues from the thin-film oven test for the 60–70, 85–100, and 120–150 penetration grades of asphalt. In general, the higher the penetration of the original asphalts the lower the percentage of retained penetration. The frequency distribution graph for the three grades is shown in figure 17. All of the 60–70 penetration asphalts retained more than 55 percent, while 10 percent of the 85–100, and 25 percent of the 120–150 penetration asphalts retained less than 55 percent. The median value for retained penetration was 66 percent for the 60–70 grade, 61 percent for the 85–100 grade and 58 percent for the 120–150 grade.

The effect of the original consistency on retained penetration is illustrated further in figure 18 for some of the asphalts of various grades from the same sources given in table 5. This graph shows that although there was a wide range in retained penetration for asphalts within each grade, there also was a fairly uniform decrease in retained penetration as the original penetration increased. These conclusions support the decision of the various specification writers to establish limits for percentage of retained penetration on a sliding scale for different penetration grades.

Softening point of thin-film residues

The softening points of the residues from the thin-film oven tests, figure 19, show considerable overlapping of the values. The overall range of the softening point of the residues for the three grades of asphalt was 111° F. to 147° F. The range for the 60–70 grade was 126° F. to 147° F., for the 85–100 grade 118° F. to 140° F., and for the 120–150 grade 111° F. to 133° F. The increase in the softening point for individual samples during the test varied considerably. Increases for the 60–70 grade varied from a minimum of 3° F. to a maximum of 17° F. For the 85–100 penetration asphalts individual increases varied from 5° F. to 17° F. The 120–150 grade showed increases varying from 3° F. to 22° F. The median increase for all grades was in the range from 8 to 10 degrees.

Ductility of thin-film residues

The distribution of the ductility of the thin-film residues for the 60–70, 85–100, and 120–150 penetration asphalts is shown in figure 20. Initially 3 of the 60–70 asphalts and 2 of the 120–150 asphalts had ductility values of less than 100 cm. There were no ductility values of less than 100 cm. for the 85–100 penetration asphalts. After the thin-film test the percentages of asphalts of each grade that had ductility values less than 100 cm. were approximately 23, 18, and 7, respectively, and 15, 7, and 0 percent of the residues had ductility values less than 50 cm.

It is of interest to note that the ductilities of many of the thin-film residues were higher than the original asphalt. Nineteen percent of the 60–70 asphalts, 42 percent of the 85–100 asphalts, and 71 percent of the 120–150 asphalts increased in ductility during heating in the thin-film test. This was accounted for by the hardening during heating that put these asphalts within the range of consistency for higher ductility.

Standard oven-loss tests

Plotted data for the standard loss tests have not been included since there was little significance to the results obtained. All of the 60–70 penetration materials had losses of less than 0.20 percent, while 97 percent of the 85–100 materials, and 89 percent of the 120–150 materials had losses of less than 0.20 percent. Maximum loss values for the 60–70, 85–100, and 120–150 grades were 0.18, 0.58, and 0.27 percent, respectively. The minimum percentages of retained penetration for the oven-loss residues for the above grades were 78, 75, and 77, respectively.

The low values for loss and the high values for retained penetration in the standard oven-loss test for all grades of asphalts covered by this report further confirms the inadequacy of the test for evaluating the hardening characteristics of asphalt cements.

Oliensis spot test

Oliensis spot tests using standard naptha were made on all of the 60–70 and 120–150 penetration grade asphalts. (Results of the spot tests on the 85–100 penetration grade asphalts were reported in part I.) Only 3 asphalts of the 60–70 and 120–150 asphalts were found to have a positive reaction. These were samples Nos. 154 (60–70 grade) and 226 (120–150 grade) from refinery 23. Sample No. 69, the 85–100 grade asphalt from the same refinery, also showed a positive spot. Asphalts from this source refined from a West Texas crude, have been known to produce positive spots even when vacuum or steam

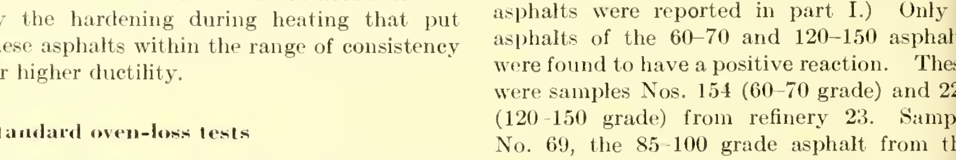


Figure 15.—Distribution of change in weight during thin-film test.

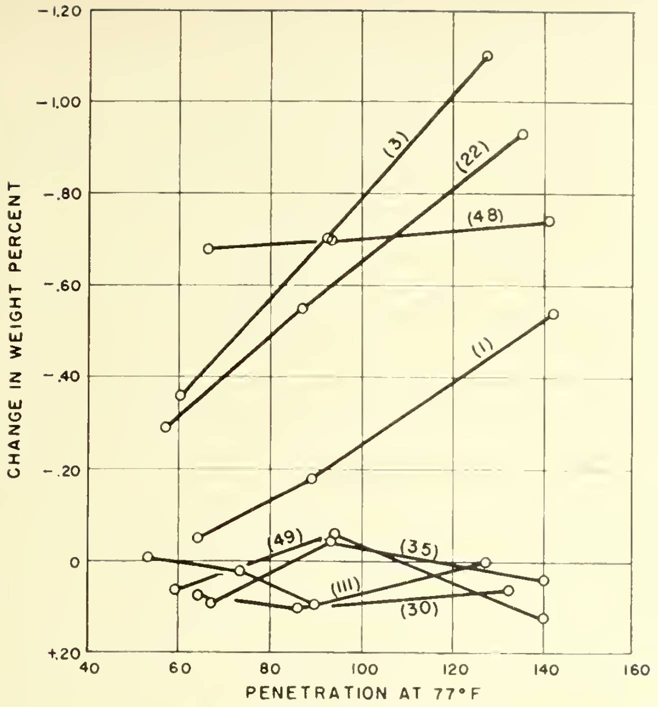


Figure 16.—Relation of penetration at 77° F. to change in weight in thin-film oven test.

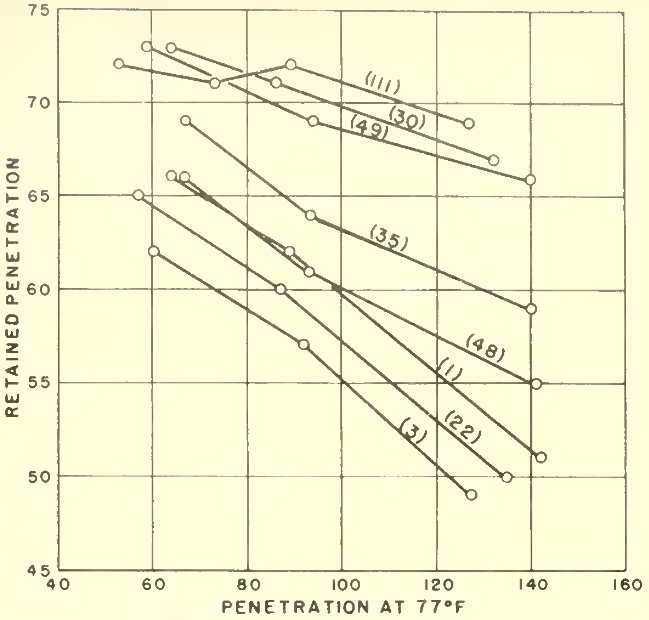


Figure 18.—Relation of penetration at 77° F. to retained penetration of thin-film residues.

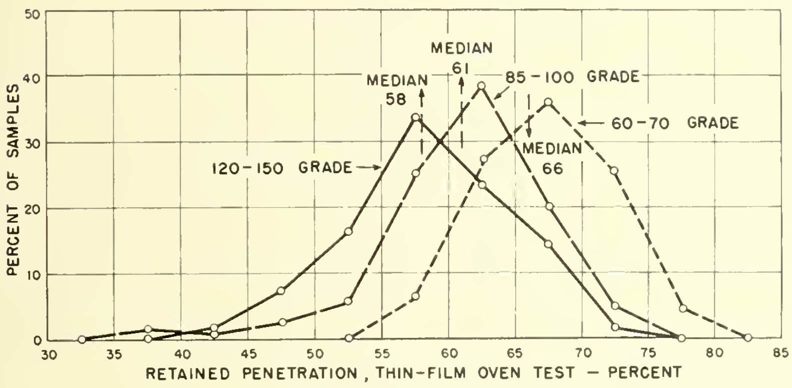


Figure 17.—Distribution of retained penetration of thin-film residues.

istillation are used. Sample No. 218 (120– 50 grade) from refinery 38 also showed a positive reaction as did sample No. 98 in the 85– 00 grade from the same source. No 60–70 sphalt was received from this producer.

Thus the results of spot tests on all grades f asphalts reported here and in part I indiate that very few of the present day asphalts ould fail such requirements in specifications, articularly if small amounts of xylene are sed with the standard naptha as is generally pecified.

General Discussion of Data

The test results reported here and in part I ere obtained primarily to provide current nformation on the properties of asphalt ements used in highways in the United States. The groups of samples in the 60–70, 85–100, nd 120–150 grades were believed to represent early all asphalts of these grades produced nd used at the time of sampling. While here may have been some changes in crude ources or methods of refining since 1955 when the samples were collected, the results of tests are believed to be indicative of the properties of asphalts produced today.

It was pointed out in part I that 12 of the 119 samples of the 85–100 penetration grade asphalts, or about 10 percent, failed to meet the standard specifications employed by the majority of the States. The test data for the other grades reported here show that 13 samples, or about 28 percent, of the 60–70 grade, and 4 samples, about 7 percent, of the 120–150 grade also were outside the required penetration limits. However, while this appears to be a rather high number of samples outside specifications, it should be noted that of the 29 samples of all grades that failed to meet the penetration range, 21 of these were within the current precision requirement for reproducibility of the penetration test.

Other than the penetration test, only 3 of the 85–100 grade asphalts failed to meet any standard AASHO specification requirement. There also were 3 samples of the 60–70 asphalts and 2 of the 120-150 asphalts that had ductility values less than 100 cm., and a single 120–150 asphalt that did not meet the requirement for solubility in carbon tetrachloride.

It was brought out in the first report that many of the current specifications, such as those of the AASHO, fail to measure the relative quality of asphalt and that asphalt materials meeting these specifications may actually show poor performance in service. Consequently during the past four years, a number of agencies have revised their asphalt specifications in an effort to raise the overall quality of asphalt. Several of the Western States followed California's lead and adopted specification requirements based on the Pensky-Martens closed cup flash test, the penetration ratio, the Furol viscosity at 275° F., and the thin-film oven test. The requirements adopted for these specifications were cooperatively developed by conferences of western producers and consumers and undoubtedly have been of value in eliminating some poor materials and in upgrading others. However, tests made by the Bureau of Public Roads show that the western requirements for the Pensky-Martens flash point, the penetration ratio and the Furol viscosity at 275° F. would not be applicable on a national basis.

Flash point requirements in specifications are intended to indicate the temperature at which asphalts may be safely heated. It was found that the presence of silicones in asphalts, which had been added during refining or storage to reduce foaming tendencies, often caused erroneous flash points by the Cleveland open cup method. The Pensky-Martens closed cup method was adopted by some of the Western States to avoid this error. The States set minimum specification limits for the 60–70, 85-100, and 120–150 penetration grades at 450° F., 440° F., and 425° F., respectively. The percentage of asphalts in the above grades that failed to meet the requirements were 9, 22, and 18.

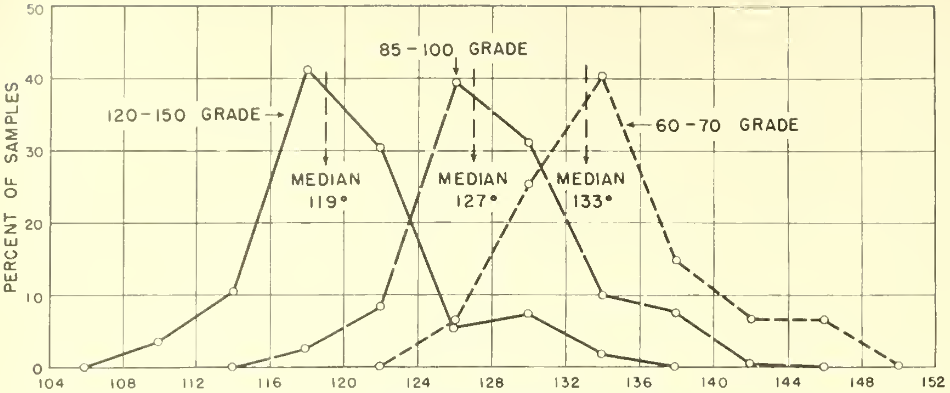


Figure 19.—Distribution of softening point of thin-film residues.

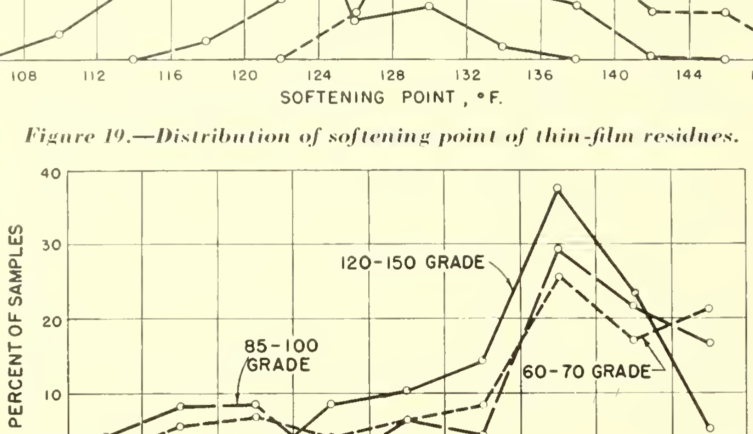


Figure 20.—Distribution of ductility of thin-film residues.

While the Pensky-Martens closed cup test may serve a useful purpose in obtaining truer flash points where silicones are present in the asphalt, the specification limits should not be set so as to exclude asphalts having good service records. Since many of the asphalts that failed the western requirements apparently were of good quality, those limits should not be adopted on a nationwide basis.

The ratio of the penetration at 39.2° F., 200 g., 5 sec., to the penetration at 77° F. 100 g., 5 sec., may serve a useful purpose in restricting the use of highly temperature-susceptible asphalts. However, the minimum requirement of 25 adopted by the Western States would have little significance if applied on a national basis. A discussion of the penetration ratios of the 85–100 asphalts in part I of the study pointed out that there was a general trend for the asphalts from the eastern sources to have the highest penetration ratios and those from western sources the lowest. The asphalts from the central areas had intermediate values. Similar trends were found for the other grades reported here. The variability in penetration ratios of asphalts throughout the country again stresses the need for the extreme caution that must be used to establish significant specification requirements.

Penetration ratios employing differences in time of penetration and loading, such as that used in the western specifications, are not believed to be true indexes of susceptibility because of the combination of test variables other than temperature. In the report on the properties of asphalts produced in the midthirties it was shown that the slope of the log-penetration temperature curve represented a useful measure of susceptibility that was more nearly related to the rheological properties of the material than any of the empirical ratios (1).[3] It is believed that if a temperature-susceptibility index is a needed requirement in asphalt specifications, consideration should be given to the use of the slope of the penetration curve or values based on fundamental measurements of consistency.

Another feature of the quality specification that serves to limit the viscosity temperature relation is the furol viscosity at 275° F. This requirement is, in effect, also a control of the temperature susceptibility of asphalt. The western specifications have established limits of 100 to 325, 85 to 260, and 70 to 210 seconds, respectively, for the 60–70, 85–100, and 120–150 grade asphalts. All of the penetration asphalts exceeded the lower limits of these requirements, but there were a number of asphalts of each grade that had viscosities higher than the maximum requirement. Approximately 30 percent of the 60–70, 8 percent of the 85–100, and 7 percent of the 120–150 asphalts had viscosities above the upper limits. Many of these asphalts are used extensively in the Eastern States and are considered to be satisfactory materials. It is again evident that if high-temperature viscosity requirements are desirable for adoption in specifications on a nationwide basis a further study should be made to establish proper limits.

To date no detailed study of the viscosity-temperature characteristics of the asphalts of this study has been made. The wide range in the viscosities values at 275° F. of the asphalts in each grade shows the necessity for giving more consideration to establishing temperatures for plant mixing and spray applications on the basis of viscosity. The difference in viscosity of the asphalts at this high temperature also indicates that there may be considerable variation in the viscosities in the temperature range to which the asphalt is subjected in the pavement. Such variations may be significant from the standpoint of the essential engineering properties.

During the past year the thin-film oven test, which was developed by the Bureau of Public Roads, received greater recognition when it was adopted by the Asphalt Institute as a specification test. The AASHO (American Association of State Highway Officials) Committee on Materials also approved the test and asphalt cement specifications have been adopted which include the thin-film oven test with suggested limits. While the Asphalt Institute specifications include requirements for retained penetration only, the AASHO specifications include the additional requirements for a maximum loss in weight and a minimum ductility of the residue. The test limits are similar to those currently in effect in a number of State specifications. Table 6 gives a summary of the test limits used by the AASHO, the Asphalt Institute and the Western States.

The limits for retained penetration suggested by the AASHO and those used by the Asphalt Institute are shown graphically in figure 21 together with the results and an analyses of the test data on the 3 penetration grades of asphalts.

The Asphalt Institute requirements fo retained penetration are slightly lower tha those in the AASHO specification for th lower penetration grades and this differenc decreases as the penetration grade increases. In order to compare these specification limits with the results of the retained penetration o the asphalts reported in this study, two sets o

Table 6.—Specification requirements for the thin-film oven test

Penetration grade	Loss in weight not more than—		Retained penetration at 77° F. not less than—			Ductility of residue at 77° F., not less than—	
	AASHO [1]	Western States [2]	AASHO	Western States	Asphalt Institute	AASHO	Western States
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Cm.*	*Cm.*
40–50	0.8	0.75	55	52	52	--------	50
60–70	.8	.8	52	50	50	50	50
85–100	1.0	.85	47	47	45	75	75
120–150	1.3	1.0	42	44	42	100	75
200–300	1.5	1.5	37	40	37	100	75

[1] Limits suggested for AASHO specification M 20–59. [2] Limits used by Western States.

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[3] Italic numbers in parentheses refer to the list of references on p. 99.

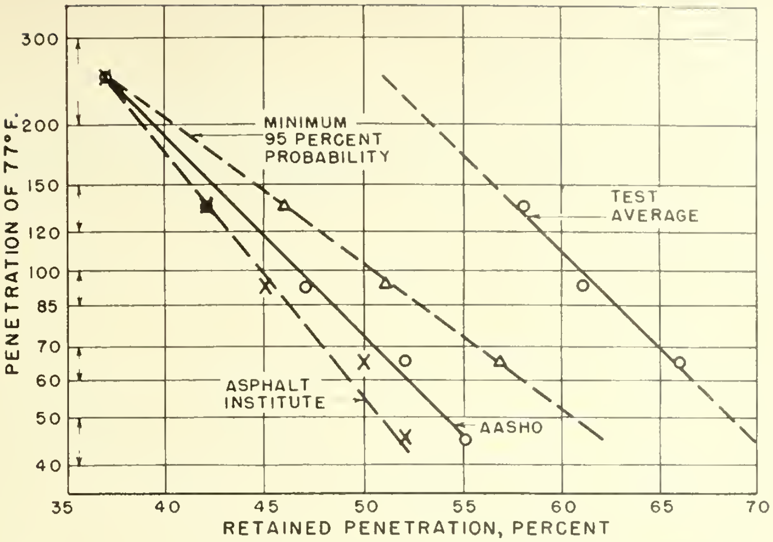

Figure 21.—Specification requirements for retained penetration, thin-film oven test.

values are also shown in figure 21. These are the average values for the 60–70, 85–100, and 120–150 grades and the minimum values established for 95 percent probability (the average − 1.96 × standard deviation), for the same grades. Although there is no particular significance to be attached to the 95 percent probability limits with respect to asphalt quality, the values are used here as a gage of the applicability of the specification limits that are proposed or in use. The Asphalt Institute limits are approximately parallel to the line shown for the average values and are about 15 percentage-points lower. The AASHO limits are nearer those established by the 95 percent probability line but are still 7 percentage-points lower in retained penetration for the 40–50 grade asphalts and the differences decrease for the higher penetration grades.

It should be emphasized that the asphalts, with possibly a few exceptions, were not produced to meet the requirements for the thin-film test. Even with this consideration there were very few asphalts that would be penalized by the AASHO limits for retained penetration which allows for differences in the reproducibility of the test. None of the 60–70 asphalts failed the requirement for retained penetration of 52 percent. Four of the 85–100 penetration asphalts were less than 47 percent, and none of the 120–150 asphalts were less than 42 percent.

The data indicate that there was no exact correlation between the amount of loss and the hardening, but where the loss was appreciable it affected hardening to a considerable degree. The relation between loss and hardening also gave some indication as to the type of hardening that occurred. Therefore, loss values are considered significant and should be determined.

All of the 60–70 asphalts had losses less than 0.8 percent, the maximum requirement suggested by the AASHO. Five of the 85–100 asphalts failed the requirement of 1.0 percent and 3 of these also failed to meet the minimum requirement for retained penetration. One sample of the 120–150 asphalts did not pass the requirement of 1.3 percent. This asphalt had a retained penetration of 42 percent, the minimum requirement of that grade.

Even though the amount of change in the ductility during the thin-film tests may not be indicative of quality, it is believed that the actual ductility of the residue is significant. Data obtained show that some asphalts retained a high percentage of penetration and yet became relatively nonductile.

The minimum requirements for ductility of the residues from the thin-film test suggested by the AASHO specification for the 60–70, 85–100, and 120–150 grades are 50, 75, and 100 cm. respectively. A minimum ductility of 100 cm. was also specified for the 200–300 grade asphalts. No requirement was specified for the 40–50 grade asphalts. Of the asphalts that did not meet the minimum ductility requirements, 6 were samples in the 60–70 grade, 17 were in the 85–100 grade, and 4 were in the 120–150 penetration grade. These are equivalent to 13, 14, and 7 percent of the asphalts included in each grade.

The most suitable limits for ductility requirements on the thin-film residues are debatable. It was pointed out in the report on the 85–100 asphalts that a minimum limit of 75 cm. appeared to be suitable for the Western States but that this requirement might be somewhat severe for the asphalts produced in the midcontinent area. The minimum requirement of 50 cm. for the 60–70 asphalts also might be somewhat severe for the asphalts produced in the midcontinent area. However, the minimum requirement of 100 cm. for the 120–150 grade appears to be satisfactory.

Summary

It is believed that the greatest value of these studies are the test results themselves. While the data reported include primarily the results of the tests used in current specifications, they serve to establish the range and medians for test values that may be expected on a national basis. Thus, they provide background information that may be used as a guide for evaluating the older specifications for asphalt cements and for judging the applicability of new tests and requirements.

It is recognized that the data offer many possibilities for analyses other than those presented here and in part I. It is believed that various trends and interrelations of the data will be of varying degrees of interest to producers, engineers, and researchers and that each may want to analyze the data in a different manner to obtain the desired information.

The analysis and discussion of the test results have served to show the inadequacy of many of the current specification requirements for asphalt cements and to illustrate the wide differences that exist in asphalts meeting the same specifications.

The need for additional studies to evaluate the significance of the differences in asphalts should be emphasized. The differences in the temperature susceptibility of asphalts have received recognition by some agencies to control plant mixing temperatures on an equi-viscosity basis. Some consideration is also being given to the effect of the differences in viscosity of asphalts on the compaction of paving mixtures on the road. However, the full significance of these differences needs to be established. The wide range in viscosities at the high temperatures raises a question as to amount of variation in viscosities of asphalts in the temperature range they are subjected to in the pavement. Research also is needed to evaluate the significance of these differences.

More information is needed concerning the hardening characteristics of asphalts. It is generally recognized that hardening during the fabrication of pavements should be limited and that this can be done effectively by the proper use of specification requirements for the thin-film oven test together with proper temperature control during plant mixing. However, studies are needed to evaluate the hardening of asphalts in pavements at road temperatures and the effect of the many variables on the hardening, such as mixture composition, design, and density of pavement.

There are some areas of research that are not a part of these studies that should be kept in mind in relation to the improvement of quality of asphalts and the development of better tests and specifications. For example, there is some concern among the States with the variability of the wetting and adhesion characteristics of asphalts. There have been some reports that modifications made to reduce hardening tendencies have resulted in poorer adhesion. This factor needs to be studied.

Asphalt technology is still based primarily on empirical tests and relations developed by trial and error methods and these will undoubtedly be continued in use for a long time. Nevertheless, the interest in developing truly fundamental data and information on rheological properties is increasing rapidly and new tools such as the Zeitfuchs capillary viscometer and the sliding plate microviscometer have been developed to make the task

(*Continued on page 99*)

Trans-Hudson River Vehicular Origin and Destination Survey

By The Port of New York Authority
Port Development Department
Planning Division

**Reported by WARREN LOVEJOY,
Transportation Economis**

A new origin and destination survey technique, called "continuous sampling," is described in this article. The big gain in this technique is the better quality of results obtained by interviewing, spread over a whole year, with approximately the same number of interviews that would be conducted in a "one-shot" survey concentrated in a small period of time. The continuous sampling technique, based on probability samples, spreads the interviewing over a long period of time and thus obtains a full variety of daily, weekly, and seasonal traffic patterns. The method is efficient and economical since it enables a small number of experienced interviewers to do all the interviewing and coding, and avoids intensive activity at any facility at any one time. In addition, the technique permits periodic analyses and provides up-to-date information, since the data are continuously obtained, and makes possible calculation of measures of sampling variability. All of these features of the continuous sampling technique tend to result in better representation of changing traffic patterns.

By using the continuous sampling technique, the Port of New York Authority, which devised the procedure, obtained statistically reliable information as to the origins and destinations of automobiles and trucks using the three major Hudson River crossings between New York City and New Jersey—the George Washington Bridge and the Holland and Lincoln Tunnels.

In 1958, a total of 80.7 million motor vehicles used the three crossings—an average daily traffic of 221,000 vehicles. Ninety-seven percent of these vehicles had at least one terminus in the 18-county New York-New Jersey metropolitan area. The largest share of weekday traffic, 34 percent, either originated in or was destined to the Manhattan central business district.

Classified by trip purposes for the year, 47 percent of the automobiles crossed the trans-Hudson facilities for work, 51 percent for recreation and personal reasons, and only 2 percent for shopping purposes. Classified according to the day of the week, automobile trips for business purposes accounted for 63 percent of the weekday trips, 24 percent of the Saturday trips, and 11 percent of the Sunday trips. Recreation and personal trips increased from 35 percent on weekdays to 73 percent on Saturdays and 88 percent on Sundays. Seasonally, the proportion of weekday business trips by automobiles varied from 57 percent in the summer to 68 percent during the winter.

On an hourly basis, during the 7–10 a.m. period 87 percent of the eastbound and 86 percent of the westbound weekday automobile trips were for work purposes. In the 4–7 p.m. period, however, only 68 percent of the eastbound trips and 72 percent of the westbound trips were for business purposes.

Traffic patterns of 1958 compared with those of 1949 are also reported in this article. The comparisons showed that traffic originating and terminating in each of the counties in the metropolitan area increased substantially over the 9-year period, with the greater percentage increases being most pronounced in the outlying counties. This and other indications appear to emphasize the growing importance of peripheral facilities and routings.

Changes in truck traffic patterns from 1949 to 1958 were less pronounced than those shown for all vehicle trips combined. Truck traffic increased only 23 percent over the 9-year period, whereas total trans-Hudson traffic increased 59 percent over the same period.

THE SURVEY TECHNIQUE

REASONABLY ACCURATE and up-to-date origin and destination information is one of the primary tools used in the planning of vehicular facilities. Since 1930, the Port of New York Authority, in connection with its planning for future vehicular facilities across the Hudson River in the metropolitan New York-New Jersey area, has conducted periodic roadside interview surveys of origins and destinations of vehicles using its tunnels and bridges. Included in the surveys were the Port Authority's vehicular facilities that connect New York City with the mainland to the west: the Holland and Lincoln Tunnels
and the George Washington Bridge, the thre
vital Hudson River crossings between th
island of Manhattan and New Jersey; an
the Outerbridge, Goethals, and Bayonn
Bridges, which connect Staten Island wit
New Jersey.

In the past, so-called "one-shot" survey
have been employed in which the traffic
1 to 3 days has been surveyed as representativ
of the entire year's traffic. These survey
were made approximately at 3-year interval
While these short-term surveys provided
substantial body of origin-destination inform
tion over the years, it was felt that improve
methods of obtaining the desired informatio
were possible. A study was consequentl
undertaken, which resulted in the adoptio
in January 1958 of a new sampling techniqu
called "continuous sampling."

Briefly, this technique is based on a car
fully designed and controlled probabilit
sample which builds up the interviews o
tained to the required number for any desire
degree of reliability by sampling over a co
siderable length of time rather than b
sampling heavily in a short period of sever
days. Thus, in the continuous survey, b
obtaining a few hundred interviews eac
day, a substantial number of interviews we
accumulated over the course of the year 195
These were obtained under all of the varyi
conditions that existed in the field througho
the year. Some of the resulting 1958 data f
the three Hudson River crossings are pr
sented later in this article. The Port Authori
has continued the sampling survey througho
1959 and in 1960.

Advantages of continuous sampling

There are a number of advantages to
derived from the use of the new traffic surv
technique. Among the more important
these, as they apply to the Port Authority
origin and destination survey problem, a
the following:

1. By spreading the sampling over a lo
period of time, this system avoids the pos
bility inherent in "one-shot" surveys th
seasonal or other variations might ma
traffic on the day or days surveyed unrep

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[1] Paul Rackow, Economic Analyst with the Port of N
York Authority, was responsible for much of the work
this survey.

ntative. In continuous sampling, a sufficient riety of traffic patterns are covered throught the year so that the survey results are presentative of normal traffic patterns.

2. Up-to-date information on origin and stination patterns is provided at any time cause the survey data are continuously ing obtained and can be analyzed periodilly, as necessary.

3. Building the size of the sample over a ng period, rather than by intensive sampling iring just a few days, has eliminated the cessity of hiring large numbers of unskilled, mporary employees as interviewers and ders. For the continuous sampling, four gularly employed interviewers did all of e interviewing as well as the coding. These en were well trained in interviewing methls, had a thorough knowledge of the geogphy of the metropolitan area, and were illed in the coding of the data.

4. Since continuous sampling avoids intene activity at any facility at any one time, ere was less chance of causing traffic constion while obtaining the origin and destinaon information. Furthermore, skilled interewers greatly reduced the time required r each interview and were thus able to get mplete information on each interview thout undue delay to the motorist.

5. By using a probability sample, it is ssible to compute mathematically the gree of reliability achieved by the survey d thus keep a constant check of the accuracy the results.

ata obtained

During 1958, a total of 92,329 interviews re obtained at six river crossings. This mber of interviews represented 0.1 percent the 85.7 million automobile and truck trips 1958 actually recorded at these crossings. hile the proportion of sampling may appear be remarkably small, the results were und to be statistically reliable.

An interview response rate of nearly 99 pernt was achieved and in all cases the full nount of information covered by the interew form was obtained. This exceptionally gh rate of response represented a conderable improvement over the level achieved the "one-shot" surveys conducted in the st, thus ensuring a high degree of reliability this phase of the survey results.

While the basic purpose of the survey was obtain information as to the origins and stinations of automobiles and trucks using e Port Authority bridges and tunnels, pplementary information was obtained conrning purpose of trip, State in which the hicle was registered, number of passengers, pe of vehicle, and time, day, and direction trip. Buses were not included in the surey, because information on bus routes was ailable from other sources.

Survey Design

The sample design for the 1958 survey had be both economical and practical in order meet the necessary restrictions as to reasonle work loads and the various working conditions which would be encountered in the field. The experience of the first year of operation (1958) and analysis of the survey results have indicated that the sample design did, in fact, meet these requirements.

There were a number of specific requirements which the survey had to be designed to accomplish:

1. Sample estimates had to be produced for a great variety of specified lines of travel with many different levels of traffic volume. A line of travel was defined as traffic moving between two geographically specified zones. These estimates had to be provided not only for the Port Authority vehicular crossings combined, but for each facility separately, on an annual basis as well as for each of the four seasons of the year.

2. The statistical population from which the sample was to be taken included all revenue (toll-paying) automobiles, trucks, and tractor-trailers using the Port Authority vehicular crossings on weekdays, Saturdays, and Sundays.

3. The sample was to be a probability sample requiring that every vehicle trip made during the year have a known probability of selection. This was to be achieved by a randomized-selection procedure at every stage of the sample design.

4. The level of reliability required in the survey specified that with the expected overall sample size of 100,000 interviews, a sample estimate of 1 percent, or 1,000 interviews, would have a coefficient of variation (maximum margin of error) of the order of 3 to 6 percent.

5. There had to be built into the sample design a procedure for developing proper measurements of the standard errors of the various sample estimates.

Development of general sample design

The general sample design had to be such that all of the statistical requirements for a probability sample were met. The design also had to accommodate the practical problems encountered in the actual conduct of the survey. These two considerations necessitated development of a number of procedures, which are outlined in the following paragraphs.

The survey budget permitted the assignment of four permanent field survey interviewers to this project. These interviewers were to be responsible for both the field work and the office work in connection with the survey. Since this was the limit of the manpower available, it was determined that the field work had to be restricted to 11 tours of duty per week.

Reasonable working hours and the necessity of minimizing fluctuations in hourly traffic volumes within shifts were used as the criteria in designating the hours for shifts, which became the primary sampling units. The shifts were set up as 8-hour periods to be spent at one facility on any one of the seven days of the week. The hours of the shifts decided upon were 11 p.m.–7 a.m., 7 a.m.–3 p.m., and 3 p.m.–11 p.m.

To insure good coverage of origin and destination patterns, directional flows, vehicle types, and traffic volumes at each facility during each 8-hour period, the design provided that the interviewer would move from one lane to another each hour in a prescribed pattern of rotation. Specified relief periods were included in each hour, but it was assumed, with good reason, that the traffic patterns during these relief periods would not be different from those sampled during the interviewing periods.

Because of the distances involved in covering the entire toll plaza of a facility, which ranged from 14 to 26 lanes, and were in some cases geographically separated, it was necessary to subdivide each facility into two or more "locations." These locations were selected in such a manner as to allow an interviewer to enumerate the number of lanes open within a location in each hour. The interviewer would rotate among the lanes at one specified location for 4 hours and then move on to another location.

Experience with previous "one-shot" surveys, supplemented by field testing, indicated that an average work load of 40 interviews per hour could be achieved by an interviewer. This does not mean that in very busy periods more than 40 interviews per hour could not be obtained. It does mean that throughout the conduct of the survey an average rate of about 40 interviews per hour was achievable.

A knowledge of hourly traffic variations at each facility on the different days of the week was utilized in selecting uniform sampling rates during each shift that would allow for reasonable work loads for the interviewers. The possibility of selecting any shift at any facility was proportionate to the amount of traffic volume expected to occur during that shift.

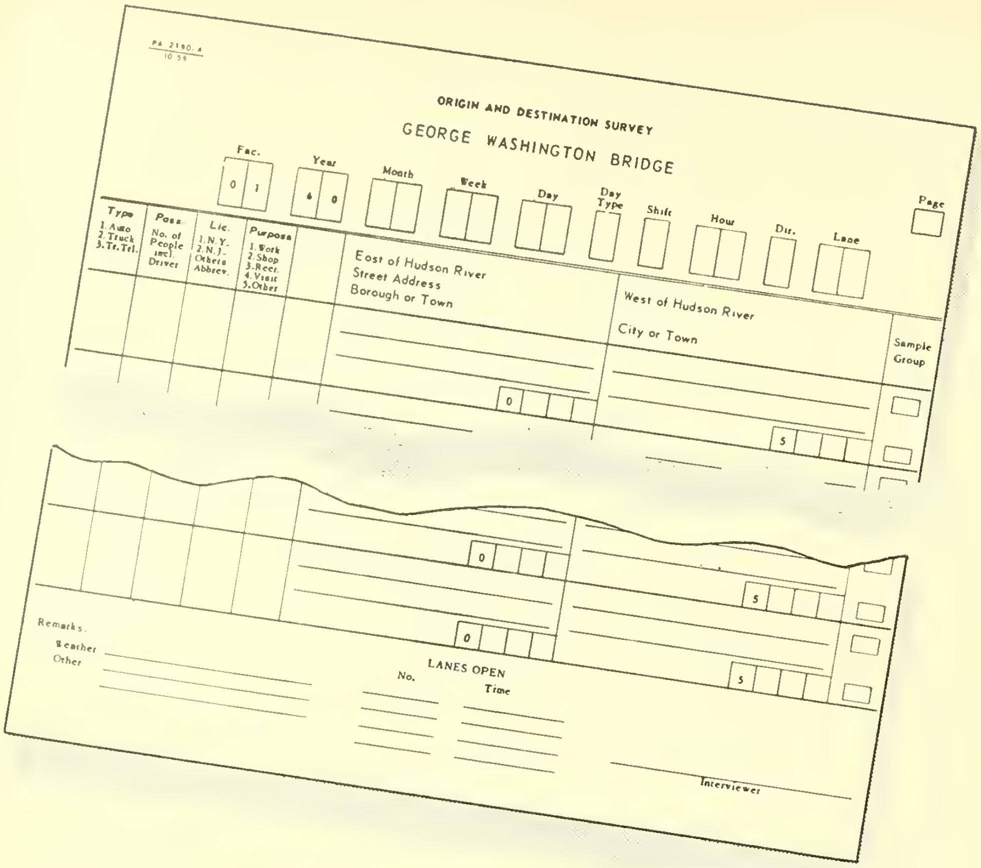
To assure a self-weighting sample, a procedure of balancing actual interviews against expected interviews was introduced. Thus, interviews could be duplicated or omitted, depending on such factors as the ratio of actual to expected lanes open, loss of interviewing time due to bad weather, and nonresponse.

A detailed description of the development of the four stages of the sample design, together with a discussion of the mathematical expressions involved, is included in appendix A. A discussion of the reliability of the results in included in appendix B.

In developing the statistical sample design the Port Authority enlisted the aid of Mr. S. T. Hitchcock and Mr. Nathan Lieder of the Bureau of Public Roads and Dr. Leslie Kish of the Survey Research Center of the University of Michigan. Dr. Kish served as statistical consultant during the entire conduct of the 1958 survey.

Field interview form

The field interview form was designed to accomplish two basic purposes: To facilitate quick and accurate recording of information obtained in the interviews; and to simplify the coding of this information for punch-card processing. After considerable field testing a form was developed which provided ample space for recording eight interviews on each sheet but also permitted a considerable amount of the required information, common to all eight interviews, to be recorded only once.



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ORIGIN AND DESTINATION SURVEY
GEORGE WASHINGTON BRIDGE

Fac. 0 1
Year 6 0
Month
Week
Day
Day Type
Shift
Hour
Dir.
Lane
Page

Type: 1. Auto 2. Truck 3. Tr. Trl.
Pass.: No. of People incl. Driver
Lic.: 1. N.Y. 2. N.J. Others Abbrev.
Purpose: 1. Work 2. Shop 3. Recr. 4. Visit 5. Other
East of Hudson River: Street Address, Borough or Town
West of Hudson River: City or Town
Sample Group

Remarks.
Weather
Other

LANES OPEN
No.
Time

Interviewer

Figure 1.—The interview questionnaire.

The top and bottom of the form are shown in figure 1. The actual form had eight sections for recording individual trip information.

THE 1958 SURVEY RESULTS

Average Daily Traffic

The average daily traffic volumes carried by the three Hudson River crossings of the Port of New York Authority—the Holland and Lincoln Tunnels and the George Washington Bridge—are shown in table 1, classified by facility, type of day, and type of vehicle. These figures, recorded as actual counts at the facilities, reached an impressive grand total of 80.7 million motor-vehicle crossings in 1958.

The 1958 origin and destination survey included automobiles and trucks using the three trans-Hudson crossings, but did not cover buses. It must be kept in mind, therefore, that the percentage figures shown in the tables and charts that follow apply to automobiles and trucks only, and cannot be applied to bus traffic.

Trip Origins and Destinations

The great majority of vehicles crossing the Hudson River in 1958 via the 3 Port Authority facilities had at least 1 terminus of their journey in the 18-county New York-New Jersey metropolitan area. This area included the 5 boroughs of New York City, 9 counties in northern New Jersey, and 4 counties in "upstate" New York and Long Island. Their locations appear in figure 2. The relative locations of the three trans-Hudson crossings are shown in figure 4 on page 92.

As shown in table 2, on weekdays 83 percent of the trans-Hudson vehicle trips were wholly within the metropolitan area, 14 percent either began or ended within the area, and only 3 percent passed through on the way to and from points outside of the area. On Sundays the intra-metropolitan area trips dropped to less than 78 percent of the total, with corresponding increases to 19 percent in trips with one terminus outside of the area and to 4 percent in the through traffic.

On both weekdays and Sundays the George Washington Bridge handled the major share of the trans-Hudson trips which had both ends outside of the metropolitan area. Presumably this reflects the role of the Ne Jersey Turnpike in handling this traffic, mo of which is oriented in a southwest-northea direction. There was a very close similari in the distribution of trips through t Lincoln and Holland Tunnels, as betwe these three basic categories of origins a destinations. The two tunnels handled ve little of the traffic passing entirely throu the metropolitan area on weekdays.

A segregation of the total trans-Huds vehicle trips into those originating or tern nating in each county, as shown in figure indicated that the Manhattan CBD (centr business district) accounted for the large share of the traffic in 1958—a third of t weekday total. It also showed that west the Hudson River three counties—Berge Hudson, and Essex—produced nearly tw thirds of the total trans-Hudson weekd trip origins and destinations.

The analysis also illustrated the significa differences that existed between weekday a Sunday traffic patterns. East of the Huds River, 49 percent of the total trans-Huds vehicle trips began or ended in Manhattan the average weekday in 1958; on Sunda however, this proportion dropped sharply 38 percent. Without exception, all of t other areas east of the Hudson River counted for larger proportions of the to traffic on Sundays than on weekdays. W of the Hudson River, the traffic patterns a shifted toward the periphery on Sunda Traffic to and from the core area, particula Hudson County, dropped sharply on Sunda whereas both the northern area and south area experienced large Sunday increases compared with their weekday share of t traffic.

Lines of travel

In order to get a reasonably accurate pict of the primary lines of travel of the 1 trans-Hudson traffic and to array the ma different trip origins and destinations cove in the survey into patterns that would aid determining the points of greatest tra Hudson traffic demand, the trips were seg gated into 30 different lines of travel betw

Table 1.—Average daily motor-vehicle traffic in 1958 at 3 Hudson River crossings, by day of week and type of vehicle[1]

Type of day at crossing	Automobiles	Trucks	Total automobiles and trucks	Buses	Total all vehicles
George Washington Bridge:					
All days	88, 840	6, 843	95, 683	1, 754	97, 437
Weekdays	84, 002	8, 966	92, 968	2, 008	94, 976
Saturdays	94, 358	2, 880	97, 238	1, 236	98, 474
Sundays	105, 695	1, 241	106, 936	1, 129	108, 065
Lincoln Tunnel:					
All days	52, 889	9, 053	61, 942	6, 236	68, 178
Weekdays	50, 343	12, 239	62, 582	6, 740	69, 322
Saturdays	58, 100	2, 665	60, 765	5, 416	66, 181
Sundays	60, 114	1, 068	61, 182	4, 766	65, 948
Holland Tunnel:					
All days	41, 523	13, 701	55, 224	357	55, 581
Weekdays	37, 581	18, 660	56, 241	347	56, 588
Saturdays	47, 976	3, 184	51, 160	412	51, 572
Sundays	53, 575	1, 954	55, 529	345	55, 874
Three crossings combined:					
All days	183, 252	29, 597	212, 849	8, 347	221, 196
Weekdays	171, 926	39, 865	211, 791	9, 095	220, 886
Saturdays	200, 434	8, 729	209, 163	7, 064	216, 227
Sundays	219, 384	4, 263	223, 647	6, 240	229, 887

[1] From actual counts at the facilities.

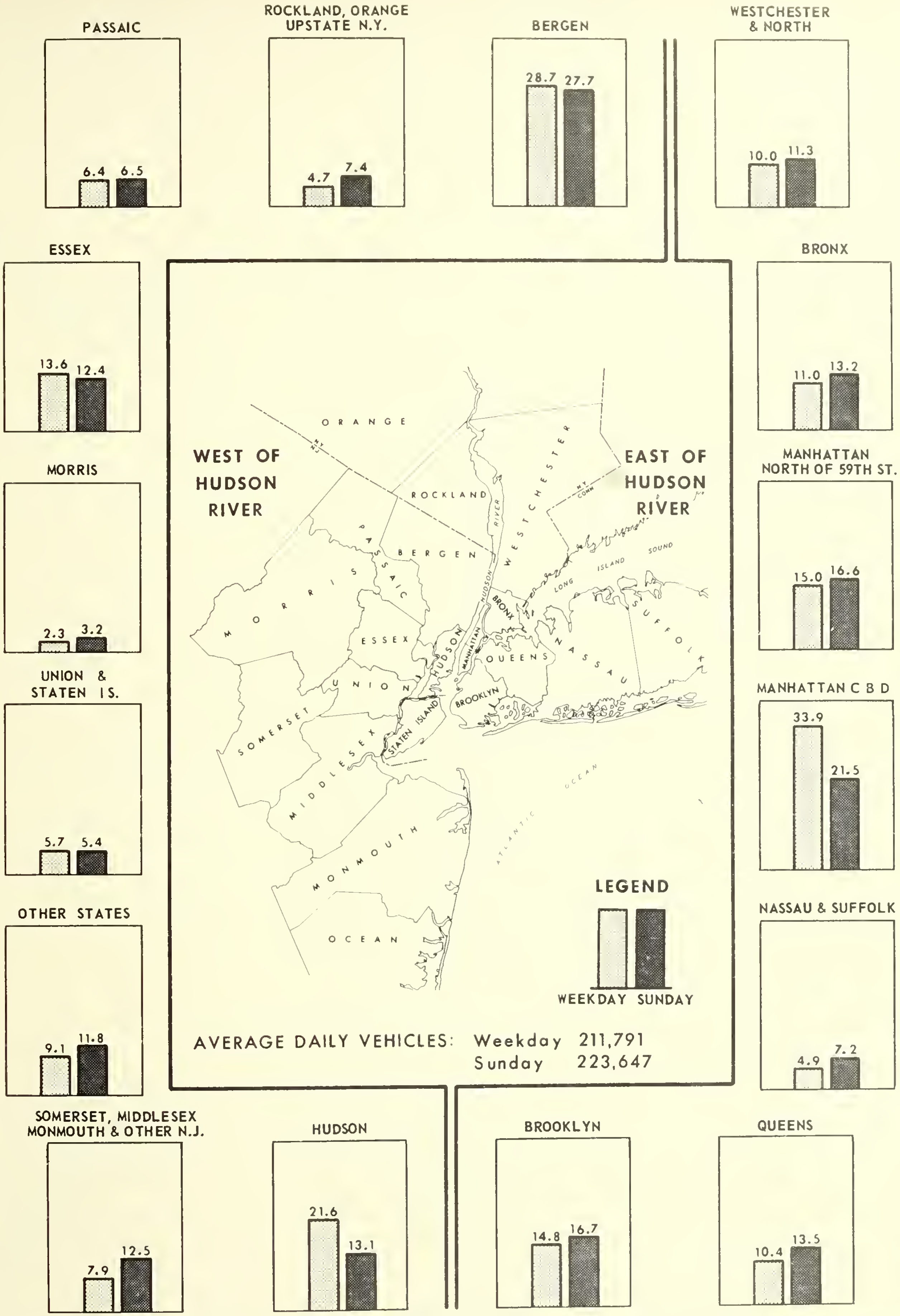

Figure 2.—Percentage distribution of origins and destinations of trans-Hudson traffic by counties on average weekdays and Sundays in 1958.

Table 2.—Percentage distribution of trans-Hudson vehicle trips in 1958 by facility and by trip origin and destination in relation to the metropolitan area

Crossing	Percentage distribution by facility		Percentage distribution according to trip end					
			Weekdays			Sundays		
	Weekdays	Sundays	Both ends in area	One end outside	Both ends outside	Both ends in area	One end outside	Both ends outside
George Washington Bridge	43.9	47.8	80.4	13.5	6.1	77.1	15.8	7.1
Lincoln Tunnel	29.6	27.4	86.0	13.6	.4	78.2	21.2	.6
Holland Tunnel	26.5	24.8	85.1	14.3	.6	77.1	22.3	.6
Three crossings combined	100.0	100.0	83.4	13.7	2.9	77.5	18.8	3.7

6 zones west of the Hudson River and 5 east of the river. This is shown in table 3.

Examination of the data indicates again the considerable difference between weekday and Sunday traffic patterns in many cases. Weekday patterns are oriented much more closely around the Manhattan CBD and the central area of New Jersey. For example, trips between the Manhattan CBD and Hudson County, which comprised 9 percent of the total trans-Hudson trips on weekdays, dropped to 4 percent on Sundays. Sunday patterns tend more toward the outlying areas, emphasizing the desirability of peripheral routes. If peripheral traffic is considered as those vehicles which have neither origin nor destination in the Manhattan CBD or in Hudson County, the peripheral traffic accounted for 53 percent of the total on weekdays and 69 percent on Sundays.

Choice of crossing

In figure 3, trans-Hudson traffic of an annual average day in 1958 has been further grouped into nine basic lines of travel in order to show how traffic over each of the nine lines was distributed among the three Hudson River crossings. The origin-destination groupings used were as follows: Northwest—Bergen, Passaic, and north; Central West—North Hudson, Essex, and Morris; Southwest—Union, South Hudson, and south; Northeast—Manhattan north of 59th St., Bronx, and north; Central East—Manhattan from Houston St. to 59th St., Queens, Nassau, and Suffolk; Southeast—Manhattan south of Houston St., and Brooklyn.

It can be seen that most of the traffic over the three east-west lines of travel (northwest-northeast, central west-central east, and southwest-southeast) used the closest facility. The choice of facility made by vehicles traveling over the diagonal lines of travel, however, reflected to a considerable extent the highways available. Thus, 66 percent of the traffic moving over the southwest-northeast line used the George Washington Bridge—the most northerly of the three crossings—and proceeded up and down the Hudson River on the west side, probably utilizing the New Jersey Turnpike. On the other hand, over the northwest-southeast line of travel, 56 percent used the George Washington Bridge and only 44 percent traveled west of the river, a reflection of the fact that there was no really good express route west of the river oriented in a northwest-southeast direction.

Truck origins and destinations

Truck traffic over the three Hudson River crossings in 1958 constituted about 18 percent of the total weekday traffic. At the Holland Tunnel approximately one-third of the weekday vehicles were trucks. Since the average truck uses as much crossing capacity as at least two automobiles, it can be said that nearly 50 percent of the Holland Tunnel weekday capacity and 30 percent of the combined weekday capacity of the three Hudson River crossings was utilized by trucks.

In view of the importance of these truck movements from a capacity standpoint, a separate analysis was made of weekday truck origins and destinations. The analysis showed a considerably greater concentration of truck origins and destinations than was the case for automobiles. East of the river, nearly 41 percent of the trans-Hudson truck trips began or ended in the Manhattan CBD, with 15 percent destined to or from the area below Houston Street, 15 percent from Houston to 34th Street, and 11 percent fro 34th to 59th Street. Another 20 perce had origins or destinations in Brooklyn an 15 percent in Queens and Long Island.

West of the Hudson River, Hudson Count was the largest generator of truck traffi nearly 36 percent of the weekday trans-Hu son truck trips started or finished in th county and Staten Island. More than ha of these trips were destined to or from th Manhattan CBD, with a fairly even split route between the Holland and Lincoln Tu nels. Approximately 15 percent of the truc trip origins and destinations west of the riv were found in Essex County, with a conce tration of these in Newark.

There is a certain amount of ambiguity these Hudson and Essex County truck tr figures. A number of over-the-road truc terminals located in those counties were us as break-bulk points or places where a chan was made from an over-the-road to a loc driver. Thus, an undetermined number truck trips which appeared to originate terminate in Hudson and Essex Counti probably were over-the-road trips or we carrying over-the-road freight. In this co nection, it is interesting to note that 20 pe cent of the truck trips over the three Po Authority Hudson River crossings in 19 had either their origin or destination ou side of the Port District, and about 3 pe cent were through trips having both orig and destination outside. Comparative fi ures for automobiles were 12 percent and percent, respectively.

The geographic distribution of truck tr movements is shown in figure 4.

Trends in Origin-Destination Patterns

To utilize origin-destination informati in planning for future vehicular needs, it necessary to obtain not only a knowled of current traffic patterns but also an und standing of the trends that have developed these patterns, particularly in the postw years. These trends can then be related shifts in the geographic concentrations population and industry which have tak place in the past within the metropolit area and are expected to occur in the futu and forecasts of future origin and destinati patterns can thus be produced.

Table 3.—Percentage distribution of trans-Hudson vehicle trips in 1958 among 30 primary lines of travel

To or from zones west of Hudsou River	To or from zones east of Hudson River											
	Brooklyn		Manhattan CBD		Upper Manhattan		Queens, Nassau, Suffolk		Bronx, Westchester, upper N.Y., New England		Total east of Hudson River	
	Weekday	Sunday	Weekday	Sunday	Weekday	Suuday	Weekday	Sunday	Weekday	Sunday	Weekday	Sunday
West Bergen, Passaic, Rockland, Orange, other north	3.2	5.2	7.7	5.2	5.0	5.2	3.4	5.1	5.4	6.1	24.7	26.8
East Bergen	1.1	1.5	4.5	2.6	4.1	3.9	1.6	2.7	3.8	4.5	15.1	15.2
Morris, Essex	2.9	2.9	6.0	3.9	1.8	2.5	2.6	3.4	2.6	2.9	15.9	15.6
North Hudson	1.6	1.1	4.3	1.8	1.0	1.0	1.6	1.2	1.4	1.1	9.9	6.2
South Hudson, Staten Island	2.9	1.3	4.7	2.3	1.1	1.0	1.8	1.4	1.6	1.4	12.1	7.4
Union, Somerset, Middlesex, Monmouth, other N.J., other States	3.5	4.9	6.3	5.3	2.0	3.5	4.2	6.7	6.3	8.4	22.3	28.8
Total west of Hudson R.	15.2	16.9	33.5	21.1	15.0	17.1	15.2	20.5	21.1	24.4	100.0	100.0

In identifying trends in trans-Hudson trip rigin-destination patterns, through comparion of the 1958 continuous survey results ith the results of the "one-shot" surveys of 949 and 1956, it was necessary to keep in nind that certain statistical variations would ccur which do not reflect basic trend changes. The following discussion of trends, therefore, is concentrated primarily on those changes that showed up over the whole period from 1949 to 1958 rather than on changes that appeared only between 1956 and 1958. Because a substantial number of vehicles crossed the Hudson River by ferry in 1949, such vehicle trips were included in the 1949 figures.

General changes

Comparison of the 1958 and 1949 traffic data, presented in table 4, showed that without exception traffic originating or ter-

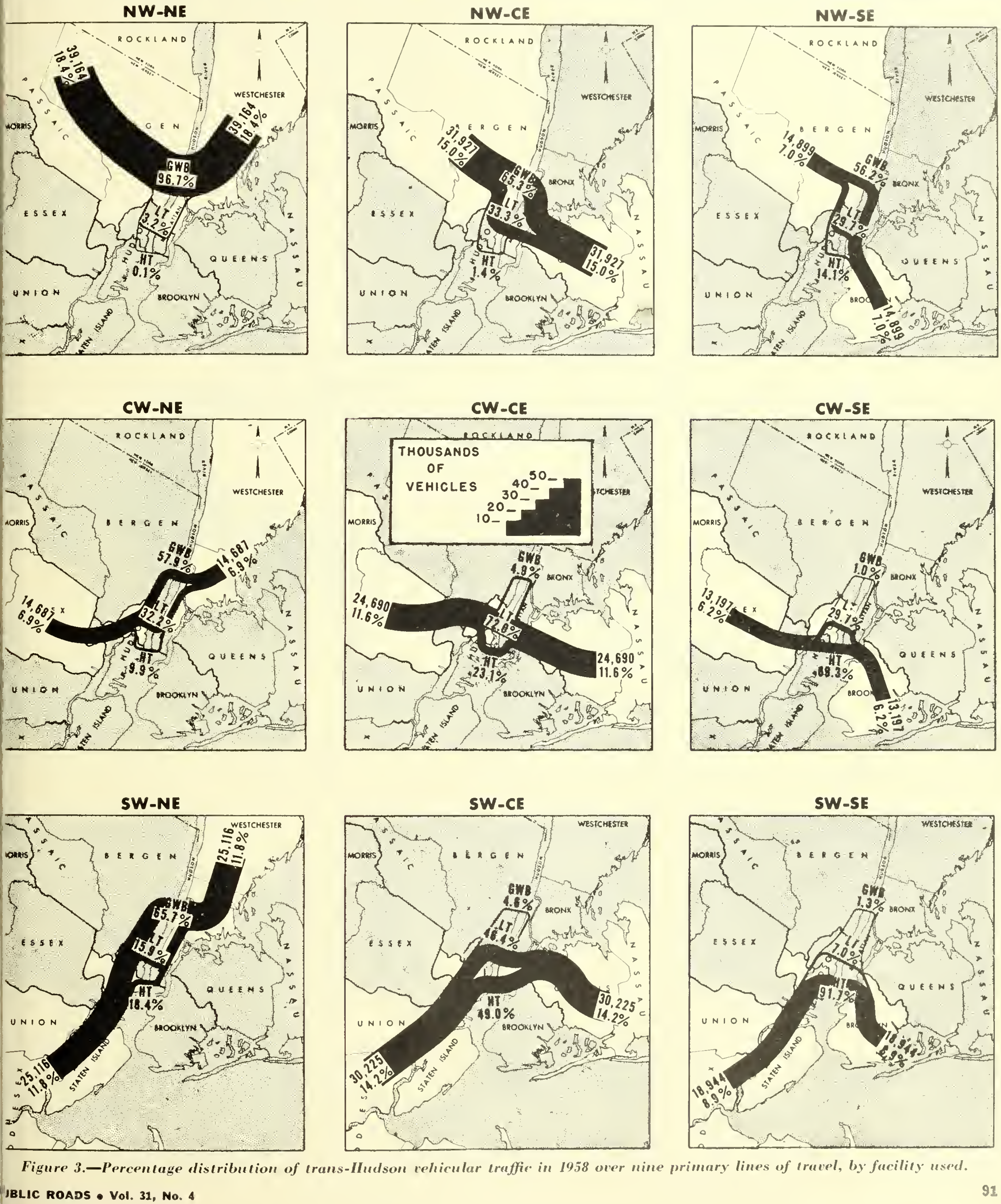

Figure 3.—Percentage distribution of trans-Hudson vehicular traffic in 1958 over nine primary lines of travel, by facility used.

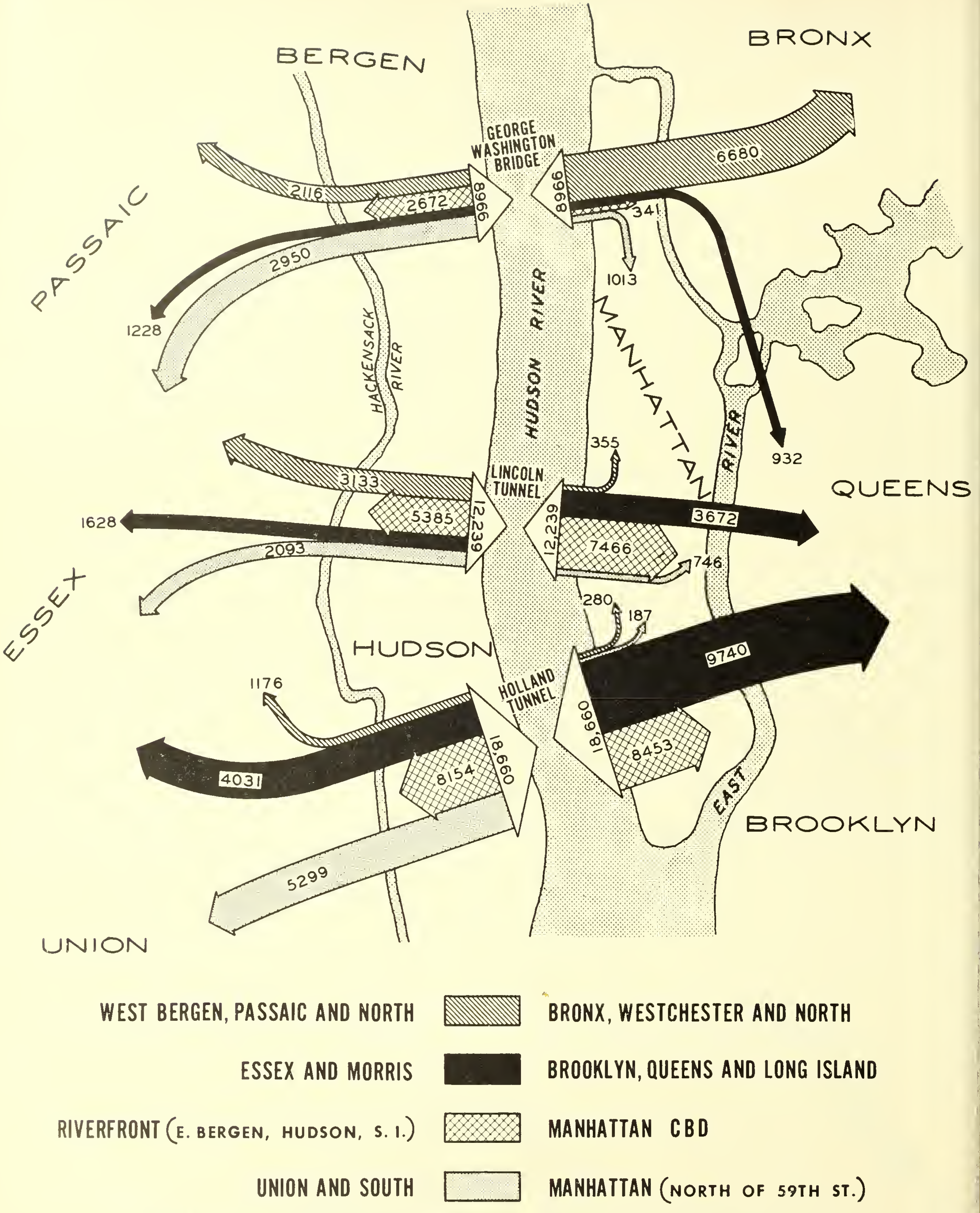

Figure 4.—Geographic distribution of trans-Hudson truck traffic on an average weekday in 1958.

Table 4.—Trends in trans-Hudson vehicular trips on weekdays and Sundays, 1919–58, according to origin and destination

Area of origin or destination	Number of vehicle trips						Percentage distribution			
	Average weekday			Average Sunday			Weekdays		Sundays	
	1949 [1]	1958	Increase [2]	1949 [1]	1958	Increase [2]	1949 [1]	1958	1949 [1]	1958
East of Hudson:			*Percent*			*Percent*				
Manhattan CBD	52, 525	71, 797	36. 7	33, 697	48, 084	42. 7	39. 3	33. 9	21. 4	21. 5
Manhattan north of 59th St.	17, 776	31, 769	78. 7	24, 363	37, 126	52. 4	13. 3	15. 0	15. 5	16. 6
Subtotal Manhattan	70, 301	103, 566	47. 3	58, 060	85, 210	46. 8	52. 6	48. 9	36. 9	38. 1
Bronx	13, 365	23, 297	74. 3	22, 939	29, 521	28. 7	10. 0	11. 0	14. 1	13. 2
Westchester and north	12, 162	21, 179	74. 1	19, 143	25, 272	32. 0	9. 1	10. 0	12. 4	11. 3
Brooklyn	20, 048	31, 345	56. 3	27, 844	37, 349	34. 1	15. 0	14. 8	17. 7	16. 7
Queens	13, 232	22, 026	66. 5	20, 883	30, 192	44. 6	9. 9	10. 4	13. 2	13. 5
Nassau, Suffolk	4, 544	10, 378	128. 4	9, 334	16, 103	72. 5	3. 4	4. 9	5. 7	7. 2
Total east of Hudson R	133, 652	211, 791	58. 5	158, 203	223, 647	41. 4	100. 0	100. 0	100. 0	100. 0
West of Hudson R.:										
Rockland, Orange, upstate N.Y	7, 752	9, 954	28. 4	16, 453	16, 550	. 6	5. 8	4. 7	10. 4	7. 4
Bergen	27, 800	60, 784	118. 6	33, 222	61, 950	86. 5	20. 8	28. 7	21. 5	27. 7
Passaic	10, 291	13, 555	31. 7	12, 182	14, 537	19. 3	7. 7	6. 4	7. 6	6. 5
Hudson	35, 016	45. 747	30. 6	22, 939	29, 298	27. 7	26. 2	21. 6	15. 7	13. 1
Essex	24, 057	28, 804	19. 7	24, 838	27, 732	11. 7	18. 0	13. 6	15. 4	12. 4
Morris	2, 673	4, 871	82. 2	5, 379	7, 157	33. 1	2. 0	2. 3	3. 3	3. 2
Union, Staten Island	7, 084	12. 072	70. 4	9, 018	12, 077	33. 9	5. 3	5. 7	5. 6	5. 4
Middlesex, Monmouth and other N.J	8, 955	16, 731	86. 8	16, 611	27, 956	68. 3	6. 7	7. 9	10. 1	12. 5
Other States	10, 024	19, 273	92. 3	17, 561	26, 390	50. 3	7. 5	9. 1	10. 4	11. 8
Total west of Hudson R	133, 652	211, 791	58. 5	158, 203	223, 647	41. 4	100. 0	100. 0	100. 0	100. 0

[1] Includes vehicles on Hudson River ferries. [2] 1958/1949.

inating in each of the counties in the etropolitan area had increased both on eekdays and on Sundays. There were, owever, marked differences in the amount of rowth produced in the different counties, articularly on weekdays and in those ounties located west of the Hudson River. n general, as might be expected from the opulation and industry trends in the metroolitan area, the traffic growth was most ronounced in the outlying counties, with the reas in the core lagging behind. This, of ourse, emphasizes the growing importance of eripheral facilities and routings, especially ince future population and industry growth n the metropolitan area is expected to be oncentrated in the outlying areas.

West of the Hudson River, Bergen, Morris, nd Union Counties and areas to the south nd west of Union all have shown greater han average increases in weekday trans-Iudson trip origins and destinations since 949. The older counties of Hudson, Essex, nd Passaic, in contrast, recorded substanially less than average growth during this eriod. This is particularly significant beause, next to Bergen County, these three lder counties have been the largest trans-Iudson traffic generators west of the river.

Although average Sunday trans-Hudson raffic has risen more slowly since 1949 than as the average weekday traffic (41 percent or Sundays as compared with 59 percent for eekdays) substantially the same differential rowth trends which occurred on weekdays n the counties west of the Hudson River also ook place on Sundays. Thus, Sunday raffic volumes to and from the outlying reas, particularly Bergen, Middlesex, Monnouth, and the south, had high growth rates hereas Hudson, Essex, and Passaic had less han average increases.

East of the Hudson River, weekday traffic o and from the Manhattan CBD has increased considerably less than the average. This, too, is significant because in 1958 the Manhattan CBD still accounted for one-third of all weekday trans-Hudson trip origins and destinations. Traffic to and from Brooklyn and Queens experienced average growth; origins or destinations in Manhattan above 59th Street, the Bronx, Westchester, and north increased at a somewhat faster than average rate; and traffic generated in Nassau and Suffolk grew at a considerably faster than average rate. As to Sunday traffic, with the exception of Nassau and Suffolk which had increases of 73 percent, all of the other counties east of the Hudson showed about an average or less than average rate of growth. It is interesting to note that in contrast to its weekday trend, the Manhattan CBD Sunday trans-Hudson origins and destinations grew at a slightly higher rate than the average amount of 41 percent over the 10-year period.

A comparison of the 1958 data with the 1956 survey data (not included in this report) showed two interesting developments which, because of their consistency in all segments of the traffic, appeared to be bona fide trend developments rather than statistical variations. In 1956, traffic to and from Orange, Rockland, and upstate New York dropped sharply as a proportion of the total trans-Hudson traffic. Presumably this was occasioned by diversions to the newly-opened Tappan Zee Bridge, the Hudson River crossing of the New York Thruway located about 15 miles north of the George Washington Bridge. In 1958, however, this segment of traffic began to rise again, both on weekdays and Sundays. It may be expected that this traffic will continue to rise as motorists become more familiar with the advantages of the Palisades Interstate Parkway route (completed August 1958), running north from the George Washington Bridge on the west side of the Hudson River.

Average Sunday trans-Hudson traffic volumes in 1958 dropped below the 1957 totals and were virtually the same as those of 1956. Comparison of the 1958 with the 1956 survey results indicated that a major reason for this poor Sunday record was a substantial drop in traffic to and from points outside of the metropolitan area to the south and west and the northeast. After increasing from 15 percent of the Sunday traffic in 1949 to 21 percent in 1956, vehicle trips originating or terminating in areas south and west of the metropolitan area declined to 17 percent in 1958. Similarly, vehicle trips destined for New England and upstate New York east of the Hudson River accounted for 6.7 percent of total Sunday trans-Hudson traffic in 1949, 7.5 percent in 1956, but only 6.5 percent in 1958. It is very likely that both the recession and the abnormally poor weather in 1958 contributed to this drop in the relatively long-haul Sunday traffic, and it is to be expected that the 1959 survey results will show a resurgence in this traffic.

Truck trip trends

Weekday trans-Hudson truck traffic increased only 23 percent from 1949 to 1958, as compared with the 59 percent growth in total trans-Hudson traffic over the same period. Changes in truck origin and destination patterns since 1949 also were less pronounced than those shown for all vehicle trips combined. The dominant fact revealed by analysis of truck trends, however, is that, despite this relative stability, the truck trends show the same tendency found in the analysis of total trans-Hudson traffic volumes: a lessening of the importance of the core areas and increased importance of the outlying areas as traffic-generators.

The distributions of truck trip origins and destinations in 1949 and in 1958 are shown in table 5. East of the Hudson River, 47 percent of the weekday truck trips originated or terminated in the Manhattan CBD in 1949, but this figure had declined to 41 percent in 1958.

Table 5.—Trends in weekday trans-Hudson truck trips, 1949–58, according to origin and destination [1]

Area of origin or destination	Percentage distribution in—	
	1949 [2]	1958
East of Hudson R.:		
Manhattan CBD	46. 5	40. 6
Manhattan, north of 59th St.	5. 4	5. 0
Bronx	7. 7	7. 7
Westchester	3. 0	3. 8
Brooklyn	19. 1	20. 2
Queens	10. 6	11. 1
Nassau, Suffolk	2. 6	4. 2
Upper N.Y. and N. Eng	5. 1	7. 4
Total east of Hudson R	100. 0	100. 0
West of Hudson R.:		
Orange, Rockland, upper N.Y.	4. 5	3. 1
Bergen	9. 7	12. 5
Passaic	6. 1	6. 0
Hudson	40. 4	35. 7
Essex	17. 1	15. 5
Morris	1. 1	1. 3
Union and Staten Island	4. 4	5. 6
Somerset, Middlesex, Monmouth	4. 3	4. 5
Other N.J	2. 7	3. 0
Other States	9. 7	12. 8
Total west of Hudson R	100. 0	100. 0

[1] The average numbers of weekday truck trips were 33,446 in 1949 and 41,234 in 1958.
[2] Includes trucks on Hudson River ferries.

Manhattan north of 59th Street also experienced a modest decline in its proportionate share of the traffic. The only areas whose share of the total traffic increased substantially were Nassau and Suffolk and upper New York and New England. West of the Hudson River, the total truck traffic originating or terminating in Hudson County declined from 40 percent in 1949 to 36 percent in 1958. The total truck traffic also dropped in Essex County, from 17 percent in 1949 to 16 percent in 1958. Bergen County, in contrast, produced less than 10 percent of the truck trips in 1949 but accounted for 13 percent in 1958.

The Tappan Zee Bridge diversions and the recession and bad weather in 1958 had much the same effect on west-of-the-river truck origin and destination patterns as was observed for the total trans-Hudson traffic. Truck trips to and from Orange, Rockland, and upper New York declined from 4.5 percent of the total in 1949 to 1.9 percent in 1956 but increased to 3.1 percent in 1958. Truck trips to and from points south and southwest of the metropolitan area rose from 12.4 percent of the total in 1949 to 19.7 percent in 1956 but dropped sharply in 1958 to 15.8 percent.

Lincoln Tunnel: Effect of third tube

The opening of the third tube of the Lincoln Tunnel in May 1957, which provided additional traffic lanes for this facility, eliminated the congestion and resulting delays which had been occurring there regularly during peak periods. The availability of this faster route through the Lincoln Tunnel resulted in both traffic-generation and diversion from other facilities, as shown by the fact that the tunnel's share of the total trans-Hudson traffic (including buses) increased from 27.7 percent in 1956 before the third tube's opening to 30 8 percent in 1958.

A special "before and after" study was made of the 1956 and 1958 survey results to measure the changes in trans-Hudson origin and destination patterns which were produced by the additional capacity at the Lincoln Tunnel. The study showed that each of the primary trans-Hudson traffic-generating areas increased its proportionate usage of the Lincoln Tunnel in 1958 but that certain of these areas experienced larger increases than others.

On an average weekday in 1958 about 41,000 automobiles, trucks, and buses destined to or from the Manhattan CBD passed through the Lincoln Tunnel. This amounted to 52 percent of the total weekday trans-Hudson vehicles with CBD origins or destinations, as compared with the 46 percent handled by the tunnel in 1956. Thus, one effect of the opening of the third tube has been to make the Lincoln Tunnel the major gateway to the central business sector of the region. As shown in table 6, the tunnel's 1958 share of the total trans-Hudson traffic (excluding buses) to and from Brooklyn, Queens, and Long Island also increased substantially over 1956. The opening of the third tube had little or no effect on the routing of traffic to or from the Bronx, Westchester, and north.

West of the Hudson River, the opening of the third tube seems to have resulted in a fairly even distribution geographically of the increases in the proportion of total traffic using the Lincoln Tunnel. All of the counties west of the river apparently were equally benefited by the elimination of delays at the Lincoln Tunnel. The primary consideration which produced variations in the amount of trans-Hudson traffic diverted to the tunnel in 1958 appears to have been the relative advantages afforded by the three-tube tunnel in serving the different points of origin or destination on the east side of the river.

Table 6.—Percentage of total trans-Hudson vehicular traffic which used the Lincoln Tunnel, 1956–58

Area of origin or destination	1956	1958
East of Hudson R.:	*Pct.*	*Pct.*
Manhattan CBD	[1] 42.2	[1] 47.5
Manhattan, north of 59th St	19.2	22.9
Bronx	5.9	5.1
Westchester	5.4	6.0
Brooklyn	17.0	22.2
Queens	32.3	39.2
Nassau	25.2	32.7
Suffolk	23.2	32.2
Upper N.Y. and N. Eng	6.5	4.2
Total east of Hudson R	26.1	29.5
West of Hudson R.:		
Orange, Rockland, upper N.Y.	6.7	11.3
Bergen	13.1	15.1
Passaic	42.9	47.1
Hudson	33.6	40.4
Essex	30.9	35.8
Morris	49.6	52.7
Union and Staten Island	25.1	26.4
Somerset	26.0	29.6
Middlesex	30.8	34.3
Monmouth	34.3	35.2
Other N.J. and other States	26.7	30.3
Total west of Hudson R	26.1	29.5

[1] If buses were included these figures would be 46 percent in 1956 and 52 percent in 1958.

Table 7.—Percentage distributions showing seasonal variations of trans-Hudson weekday vehicular trips in 1958 among 30 primary lines of travel

		To or from zones east of Hudson River					
To or from zones west of Hudson River	Season[1]	Brooklyn	Manhattan CBD	Manhattan north of 59th St.	Queens, Nassau, Suffolk	Bronx, Westchester, upper N.Y., New England	Total east of Hudson River
West Bergen, Passaic, Rockland, Orange, other north	1st	2.8	8.1	5.1	3.1	5.3	24.4
	2d	3.6	7.7	5.1	3.4	5.5	25.3
	3d	3.6	7.5	4.6	3.5	5.4	24.6
	4th	2.8	7.6	5.2	3.4	5.3	24.3
	Year	3.2	7.7	5.0	3.4	5.4	24.7
East Bergen	1st	1.0	5.4	4.5	1.3	4.0	16.2
	2d	1.2	4.7	3.8	1.7	3.6	15.0
	3d	1.1	4.0	3.9	1.8	3.6	14.4
	4th	1.1	4.2	4.3	1.6	4.0	15.2
	Year	1.1	4.5	4.1	1.6	3.8	15.1
Morris, Essex	1st	2.9	6.5	1.9	2.2	2.2	15.7
	2d	2.8	6.0	1.7	2.9	2.8	16.2
	3d	2.9	5.5	1.8	2.8	2.7	15.7
	4th	3.0	6.2	1.6	2.6	2.5	15.9
	Year	2.9	6.0	1.8	2.6	2.6	15.9
North Hudson	1st	1.9	4.5	1.3	1.7	1.3	10.7
	2d	1.6	4.1	.9	1.4	1.4	9.4
	3d	1.4	4.0	.9	1.5	1.5	9.3
	4th	1.7	4.4	1.1	1.7	1.4	10.3
	Year	1.6	4.3	1.0	1.6	1.4	9.9
South Hudson, Staten Island	1st	3.0	5.1	1.4	1.8	1.6	12.9
	2d	2.7	4.6	1.1	2.0	1.4	11.8
	3d	2.7	4.7	1.0	1.9	1.5	11.8
	4th	3.1	4.8	1.1	1.6	1.7	12.3
	Year	2.9	4.7	1.1	1.8	1.6	12.1
Union, Somerset, Middlesex, Monmouth, other N.J., other States	1st	3.2	6.5	1.8	4.0	4.6	20.1
	2d	3.2	6.5	2.2	4.3	6.1	22.3
	3d	4.1	6.1	2.1	4.6	7.3	24.2
	4th	3.2	6.3	1.9	3.9	6.7	22.0
	Year	3.5	6.3	2.0	4.2	6.3	22.3
Total west of Hudson R.	1st	14.8	36.1	16.0	14.1	19.0	100.0
	2d	15.1	33.6	14.8	15.7	20.8	100.0
	3d	15.8	31.8	14.3	16.1	22.0	100.0
	4th	14.9	33.5	15.2	14.8	21.6	100.0
	Year	15.2	33.5	15.0	15.2	21.1	100.0

[1] The seasons and the average weekday trips in each are: 1st, Jan.–Mar., 181,670 trips; 2d, Apr.–June, 217,545 trips; 3d, July–Sept., 230,889 trips; 4th, Oct.–Dec., 216,332 trips; annual average, 211,791 trips.

Seasonal, Daily, and Hourly Traffic Variations

Information concerning average annua origin-destination patterns is important fo many planning purposes. It is just as important, however, to obtain a knowledge o peak period patterns, which provide the basi for the development of design criteria upo which the physical plans for vehicular faciliti are based. Seasonal and hourly variations ar two important indicators of peak condition One of the great advantages of a continuou sampling type of origin-destination survey i its ability to develop such information.

Seasonal variations

The 1958 survey was designed to provid seasonal data by dividing the survey perio into four 13-week quarters approximatel equivalent to the four seasons of the year Framed within the calendar year, January-March was labeled the first season, April-June the second, July–September the third and October–December the fourth. Analysi showed that seasonal variations did, in fact exist, particularly in traffic to and fron certain areas. The analysis also confirme the previous belief that the first and thir seasons (winter and summer) represent th extremes in seasonal variations in origin destination patterns and that the second an fourth seasons (spring and fall) are ver similar to each other and generally fairl representative of the average annual pattern

The size of the 1958 sample on weekdays nd the grouping of the many trips into the 0 primary lines of trans-Hudson vehicular ravel provided sufficient data so that statistially reliable origin-destination information vas obtained for each season of the year. 'he sample size for Sundays, however, while ufficient to yield reliable data on annual unday patterns, was too small to permit a ne analysis of Sunday traffic patterns in ach season. The analysis, therefore, was oncentrated primarily on weekday seasonal ariations. It is expected that combining the 959 origin-destination survey Sunday data vith the 1958 Sunday data will permit a eliable analysis of Sunday seasonal variations.

As shown in table 7, traffic to and from Ianhattan accounted for 52 percent of the otal trans-Hudson weekday vehicle trips ast of the Hudson River during the winter first) season but dropped to 46 percent during he summer (third) period. The area of the 3ronx and north, on the other hand, experinced an increase in its share of the total raffic from a winter proportion of 19 percent o 22 percent in the summer. Traffic to and rom Brooklyn, Queens and Long Island also howed modest proportional gains in summer.

West of the river, the largest variation was bserved in trips to and from Union County nd the south, which accounted for 24 perent of all trans-Hudson trips in the summer ut only 20 percent in the winter. Trips etween this area and the area of the Bronx nd north (east of the Hudson River) experinced the largest seasonal variation, producng 7.3 percent of the summer traffic but only .6 percent in winter. The areas adjacent to nd west of the Hudson River—East Bergen, Vorth and South Hudson, and Staten Island—ll produced a smaller share of the total traffic the summer than in the winter months. 'here was little noticeable seasonal variation the other areas west of the river.

Further study of the data showed clearly hat traffic to and from Union County and he south varied seasonally even more on undays than on weekdays. Vehicles with rigins or destinations in this area accounted or almost one-third of the total trans-Hudson rips on summer Sundays as compared with pproximately one-quarter of the winter Sunay trips. Sunday traffic between this area nd the Bronx and north produced 9.5 perent of the summer totals and only 7.2 percent f the winter Sunday volumes.

It is presumed that the primary cause of hese observed seasonal variations is that the listribution between business and nonbusiess trips changes markedly from season to eason. These changes in purpose of trip are hown later in this report. Trips with a usiness purpose tend to be concentrated in he core of the metropolitan area, and it is hese areas which account for a higher proporion of the trans-Hudson trips in the winter nd nonsummer months. Nonbusiness trips, n the other hand, are oriented toward the nore peripheral areas which, as a result, how higher proportions in the summer when onbusiness trips reach their highest level. Thus, it follows that the summer months and weekends will produce the peak conditions which peripheral facilities must be designed to handle but that facilities serving the core areas should be designed for commuter peaks during the nonsummer months.

Table 8.—Percentage distribution of weekday trans-Hudson automobile trips in 1958, showing peak-hour origin-destination patterns

Area of origin or destination	Both directions, 24 hours	Predominant direction		Reverse direction	
		7–10 a.m. eastbound	4–7 p.m. westbound	7–10 a.m. westbound	4–7 p.m. eastbound
East of Hudson R.:					
Bronx, Westchester and Manhattan north of	21.3	21.9	21.5	25.4	23.2
59th St	17.7	18.4	18.4	17.8	20.7
Manhattan CBD	31.8	39.3	38.9	17.5	24.1
Brooklyn, Queens, Long Island	29.2	20.4	21.2	39.3	32.0
Total east of Hudson R.	100.0	100.0	100.0	100.0	100.0
West of Hudson R:					
Rockland, Orange, Passaic, West Bergen, upstate N.Y	26.7	31.1	30.9	30.0	28.4
Essex, Morris	15.7	12.4	13.3	15.4	17.2
Union, south	21.4	13.3	17.5	19.5	20.1
East Bergen, Hudson, Staten Island	36.2	43.2	38.3	35.1	34.3
Total west of Hudson R	100.0	100.0	100.0	100.0	100.0

Hourly variations

Analysis of the weekday origin-destination data for 1958 by hour of the day and by direction indicated that in the future more attention must be focused on peak-hour origin and destination patterns, particularly on the east side of the Hudson River. The survey revealed that large differences existed between the average pattern for an entire weekday in both directions and the patterns which occur during peak periods in each direction. This is shown, for automobile trips only, in table 8.

On an average weekday in 1958 about 32 percent of the trans-Hudson automobile trips originated or terminated in the Manhattan CBD. During the peak periods in the predominant direction (7–10 a.m. eastbound and 4–7 p.m. westbound) this figure rose to 39 percent. Conversely, for the reverse direction during the morning peak period, westbound from 7–10 a.m., only 18 percent of the trans-Hudson automoblies originated in the CBD, and in the reverse direction during the evening peak period, 4–7 p.m. eastbound, only 24 percent terminated in the CBD.

Just the opposite trend is evident for Brooklyn and Long Island traffic. On an average weekday, this area accounted for 29 percent of the total trans-Hudson automobile trips. However, in the predominant directions of travel—eastbound in the morning peak period and westbound in the afternoon peak—Brooklyn and Long Island accounted for only about 21 percent of the total. In the reverse direction, 7–10 a.m., this area accounted for 39 percent of the westbound trans-Hudson automobile trips, and in the evening peak, 4–7 p.m., 32 percent of those eastbound. Hourly variations in traffic to and from upper Manhattan and the area of the Bronx and north were much less pronounced, although the reverse flow in some instanees, especially the 7–10 a.m. westbound movement to and from the Bronx and north, accounted for a higher proportion of the total traffic than was the case for the entire 24 hours.

These variations clearly indicate the necessity of using peak origin and destination figures rather than average daily figures in planning ramps or approaches on the east side of the Hudson River. The tremendous differences in directional origin and destination patterns during the peak hours also suggest the possibility of utilizing reversible lanes on these ramps or approaches. Finally, the peak patterns show that the bulk of the New York automobile trips to New Jersey in the peak periods originate in areas outside of Manhattan whereas the majority of the peak-period New Jersey automobile trips are destined for Manhattan.

West of the Hudson River, the hourly origin and destination patterns did not generally vary as substantially from the 24-hour averages as they did east of the river. It is evident, however, that the area including Rockland, Orange, Passaic, West Bergen, and the north accounted for a higher proportion of the peak traffic volumes in both directions than it did over the entire 24 hours. In the predominant direction peak periods, eastbound in the morning and westbound in the evening, Essex and Morris Counties and the area of Union County and the south both generated a lower proportion of the trans-Hudson automobile trips than they did during the rest of the day. A plausible explanation is that relatively good commuter rail service exists in these areas. It is also worth noting that East Bergen and Hudson Counties, lying along the Hudson River, produced 43 percent of the eastbound 7–10 a.m. trans-Hudson automobile trips. This compares with a 24-hour figure for this area of 36 percent.

Purpose of Automobile Trips

The experienced interviewers used during the continuous sampling survey were able, in virtually all cases, to obtain complete information from each of the motorists interviewed. Thus, reliable survey data were obtained covering such travel characteristics as purpose of trip, occupants per vehicle, and residence (State) or license plate of each vehicle. Analysis of these data can contribute materially to an understanding of the various competitive factors that produce trans-Hudson travel and particularly factors motivating the choice of mode of transportation.

A question as to purpose of trip was included in the survey in order to obtain information concerning forces that generate trans-Hudson

Table 9.—Percentage distribution of trans-Hudson automobile trips by trip purpose

Trip purpose	Weekdays	Saturdays	Sundays	All days
George Washington Bridge:				
Work	62.4	21.7	9.2	46.1
Recreation and other	35.7	75.9	90.3	52.2
Shopping	1.9	2.4	.5	1.7
Lincoln Tunnel:				
Work	60.5	23.5	14.0	46.1
Recreation and other	37.1	72.7	85.4	51.6
Shopping	2.4	3.8	.6	2.3
Holland Tunnel:				
Work	67.8	27.1	10.0	50.1
Recreation and other	30.6	70.0	88.2	48.0
Shopping	1.6	2.9	1.8	1.9
Three crossings combined:				
Work	63.1	23.5	10.8	47.0
Recreation and other	34.9	73.5	88.3	51.1
Shopping	2.0	3.0	.9	1.9

automobile traffic volumes. The results, shown in table 9, indicate that of the total automobile trips during 1958 across the three Port Authority trans-Hudson facilities, 47 percent were made for business purposes, 51 percent were made for recreation and personal reasons, and a surprisingly small 2 percent for shopping purposes.

Of course, these figures varied according to the day of the week and the season of the year. Trips for work purposes accounted for 63 percent of the weekday trips, 24 percent of the Saturday trips, and 11 percent of the Sunday trips. Recreation and personal trips increased from a low of 35 percent on weekdays to 73 percent on Saturdays and 88 percent on Sundays. Shopping trips accounted for 2 percent on weekdays, 3 percent on Saturdays, and 1 percent on Sundays. Seasonally, the proportion of weekday trips made for work purposes varied from 68 percent during the winter months to 57 percent in the summer.

There were also variations in purpose of trips according to the trans-Hudson crossing used, particularly on weekdays. Nearly 68 percent of the weekday automobile trips through the Holland Tunnel had work as their purpose, compared with 62 percent for the George Washington Bridge and 60 percent for the Lincoln Tunnel. On Sundays 14 percent of the automobile trips through the Lincoln Tunnel had work as their purpose while only 10 percent of the Holland Tunnel and 9 percent of the George Washington Bridge trips were in connection with work. For each of the three facilities, the proportion of trips made for shopping, whether weekday or weekend, never rose above 3 percent.

On an hourly basis, the survey showed that the usual assumption that most automobile trips made between 7 and 10 a.m. and 4 and 7 p.m. are work trips was substantially true in the morning but not in the afternoon. As shown in the upper half of table 10, during the 7–10 a.m. period 87 percent of the eastbound and 86 percent of the westbound weekday automobile trips were for work purposes. In the 4–7 p.m. period, however, only 68 percent of the eastbound trips and 72 percent of the westbound trips were for business purposes. The afternoon peak-hour volumes in both directions were swelled by a considerable amount of recreation and personal traffic. This was particularly true of the eastbound movement, and when this fact is related to the afternoon peak origin and destination information already described, it becomes apparent that substantial numbers of New Jersey people are attracted to New York during these hours by the cultural and recreational activities in Manhattan.

State of Automobile Registration

In order to acquire additional information as to the sources of trans-Hudson automobile traffic-generation, the State in which each automobile was registered, as shown on the license plate, was recorded. Approximately 50 percent of all of the automobiles crossing the Hudson River on Port Authority facilities on weekdays carried New Jersey license plates. Nearly 41 percent had New York plates and

Table 10.—Number and percentage distribution of weekday peak-hour trans-Hudson automobile trips in 1958 by purpose of trip and by State in which automobile was registered

Origin—purpose and State	Predominant direction				Reverse direction			
	7–10 a.m. eastbound		4–7 p.m. westbound		7–10 a.m. westbound		4–7 p.m. eastbound	
	Number	Percentage	Number	Percentage	Number	Percentage	Number	Percentage
Purpose of trip:								
Work	15,697	87.4	13,616	71.7	12,930	86.2	13,639	68.4
Recreation and other	2,012	11.2	4,918	25.9	2,010	13.4	6,022	30.2
Shopping	251	1.4	456	2.4	60	.4	279	1.4
Total	17,960	100.0	18,990	100.0	15,000	100.0	19,940	100.0
State of registration:								
New Jersey	13,973	77.8	13,236	69.7	2,925	19.5	6,122	30.7
New York	3,179	17.7	4,292	22.6	11,010	73.4	12,303	61.7
Other	808	4.5	1,462	7.7	1,065	7.1	1,515	7.6
Total	17,960	100.0	18,990	100.0	15,000	100.0	19,940	100.0

the remaining 9 percent were from all ov
the country. On Sundays the proportion
New Jersey cars dropped to 46 percent, an
New York and other cars rose to 43 percen
and 11 percent, respectively.

New Jersey residents predominated amon
users of the Lincoln Tunnel both on weekday
and Sundays, as shown in table 11. Ne
Jersey also had a high proportion of autom
biles using the George Washington Bridg
on weekdays but a majority of New York an
other cars used the facility on Sundays. U
of the Holland Tunnel on weekdays was mor
evenly balanced than the other two facilitie
Surprisingly enough, however, on Sunday
more New Jersey than New York cars use
the Holland Tunnel, probably because th
New York commuters to New Jersey wh
make up a large part of the Holland Tunn
weekday traffic volumes are missing o
Sundays.

Analysis of the license plate data by hou
and direction yielded important informatio
on the characteristics of trans-Hudson aut
mobile travel during the commuter hours, a
shown in the lower half of table 10. In th
predominant direction, eastbound 7–10 a.m
and westbound 4–7 p.m., New Jersey cars wer
in the majority although this was more tru
of the morning traffic (78 percent of the total
than in the afternoon (70 percent). Th
actual number of New Jersey automobile
moving eastbound in the morning was ver
close to the number moving westbound in th
evening, indicating that the bulk of th
traffic was composed of New Jersey resident
commuting to and from New York. In con
trast, however, over 30 percent more Ne
York cars traveled westbound in the evenin
peak than eastbound in the morning peak
This imbalance was even more pronounce
for the cars from other States.

In the reverse direction, New York ca
predominated during the peak hours althoug
again this was more true in the morning tha
in the evening peak. The 4–7 p.m. east
bound automobile traffic was augmented b
a substantial number of New Jersey cars, mo
of which can be assumed from the precedin
analysis on purpose of trip to be nonbusine
vehicles. As a result, the number of aut
mobiles crossing the Hudson River eastbour
during the 4–7 p.m. peak outnumbered th
morning westbound flow by nearly one-thir

Persons Per Automobile

The average automobile crossing the Hudso
River via the three Port Authority crossings i
1958 carried the driver and one passenge
The number of persons per automobile varie
considerably from the annual average figur
of 2.05, however, depending on the day of th
week, hour of day, direction of trip, an
facility used. The average load was 1.8
persons per automobile on weekdays, but ros
to 2.39 on Saturdays and 2.75 on Sundays.

Contrary to general impression, averag
loads during rush hours in the predominan
direction (7–10 a.m. eastbound and 4–7 p.n
westbound) tended to be somewhat less tha
nonrush-hour loads. Thus, at the Georg

Table 11.—Percentage distribution of trans-Hudson automobile trips in 1958 according to State in which automobile was registered

Crossing	Weekdays			Sundays		
	New Jersey	New York	Other	New Jersey	New York	Other
George Washington Bridge	49.6	40.5	9.9	40.2	47.2	12.6
Lincoln Tunnel	56.1	35.4	8.5	52.5	36.7	10.8
Holland Tunnel	44.4	48.0	7.6	49.8	39.8	10.4

Washington Bridge there was an average of 1.69 persons per automobile crossing eastbound between 8 and 9 a.m. compared with a nonrush-hour eastbound figure of 1.92. Similarly, at the Holland Tunnel an average of 1.70 persons per car traveled eastbound from 8 to 9 a.m. compared with 1.83 for the nonrush hours. The Lincoln Tunnel was the only crossing where the average rush-hour load, 1.91 persons per automobile eastbound, reached a level comparable with the nonrush-hour figure of 1.94. Presumably this is the effect of car pooling.

There is quite obviously a greater tendency for "blue-collar workers" commuting to the ndustrial areas in New Jersey to utilize car pools. Thus, at each Hudson River crossing, the westbound load per automobile from 7 to 8 a.m. (the peak morning hour westbound) was substantially higher than the nonpeak loads. For the three Port Authority crossings combined, there was an average of 2.04 persons per car from 7 to 8 a.m. westbound, higher than the 1.85 westbound for the nonrush hours. The fluctuations are shown in figure 5.

Appendix A

Statistical Model

The sample structure as finally designed was a four-stage probability sample in which the probability of selection of any vehicle in the universe was the product of the probabilties of selection in each stage. With this method the chance of selection of a vehicle at any one facility was the same as at any other facility. Thus, the survey results from each facility could be combined for analysis purposes. The four stages of selection were:

Shift.—An 8-hour work period at one of the facilities on a day of the week.

Location.—A geographically contiguous group of lanes at a facility at which the interviewer remains for 4 hours.

Lane.—One of the toll lanes within a ocation.

Vehicle.—An automobile, truck, or tractor-trailer passing through a lane. (Buses were excluded from the survey.)

A total of 572 shifts per year, or 11 for each of the 52 weeks, were chosen. It was estimated that an annual total of about 100,000 nterviews would be obtained, but only 92,329 were actually made, due primarily to ess than expected traffic through the facilities n 1958. The interviews covered 0.1 percent of the total of 85.7 million vehicles which crossed the facilities in 1958.

Shifts were selected in groups of two seasons each or 286 shifts for each half year. There were three basic criteria which had to be met: (1) Stratification to represent the different types of shifts properly and spread them as evenly as possible throughout each season; (2) random selection of shifts; and (3) roughly equal work loads per shift.

The first step in selecting the shifts (primary sampling units) to be covered was to construct a selection table. The table set down in orderly sequence a group of 84 strata consisting of each of the three tours of duty on each of the seven days of the week at each of the four facilities (the three Staten Island bridges were handled as one); thus, $3\times7\times4=84$.

Each stratum was represented by the measure f_i which was proportional to the expected hourly density of traffic flow in thousands for that stratum. Each f_i was made to be an integer for ease of selection and balance. The sum of the f_i's for all strata was 286, divided into 143 for each of the 2 seasons. Systematic random sampling was used on the selection table to make up 13 weekly work loads of 11 shifts each, with each work load roughly spread across the strata.

The probability Pr of selecting a vehicle crossing a facility may be represented as:

$$Pr=\left(\frac{11f_i}{286}\right)\left(\frac{1}{24f_i}\right)\left(\frac{44}{60}\right)=\frac{1}{851}$$

The first term denotes the probability of selecting a shift. This was the first stage of selection. Eleven shifts were selected each week with probability $f_i/286$.

The third term reflects the fact that interviewing takes place only 44 minutes out of each hour, the balance of each hour being given to relief.

The middle term represents a composite of the last three stages of selection: Location, which is the second stage; lane within location, which is the third stage; and vehicle within lane, which is the fourth or ultimate stage of selection. Thus, Pr (selection within a shift)$=1/24f_i=Pr$ (location) $\times$ Pr (lane within location) $\times$ Pr (vehicle within lane).

Therefore the probability or the rate for selecting a vehicle within a lane may be expressed as:

$$\frac{1}{24f_i}\times\frac{1}{Pr\text{ (location)}}\times\frac{1}{Pr\text{ (lane within location)}}=$$

$$Pr\text{ (vehicle within lane)}=\frac{1}{t}$$

where t is the sampling interval for a stated hour in the selected lane and is equal to the product of the denominators of the three expressions.

For example, at a facility where there were two locations and four lanes were expected to be open during a specified hour at the selected location, Pr (location) $=\frac{1}{2}$ and Pr (lane within location) $=\frac{1}{4}$. The sampling interval t is equal to $24f_i\times\frac{1}{2}\times\frac{1}{4}=3f_i$. If f_i for this shift was 4, then the interviewer would select every twelfth vehicle.

Locations were selected by random procedures and balanced for relevant factors over a season. Lane rotation systems at each location were defined for all facilities in advance. Fractional intervals for selection of vehicles within lanes were made into integers by a balanced process of randomization. At all stages of selection randomized procedures were always used and carefully specified.

Derivation of tables

In order to simplify the use of the tables contained in this report, where only percentages were desired, it was decided to use proportions derived from the sample estimates obtained from the punch-card tabulations of the survey results. The estimates used in this report are thus ratio estimates of the form, $p=x/n$

where: $x=$the total "self-weighting" card count of all vehicles with an "x" characteristic.

$n=$the card count in the entire sample or some subclass.

These proportions can be used to estimate numbers of vehicles for any desired character-

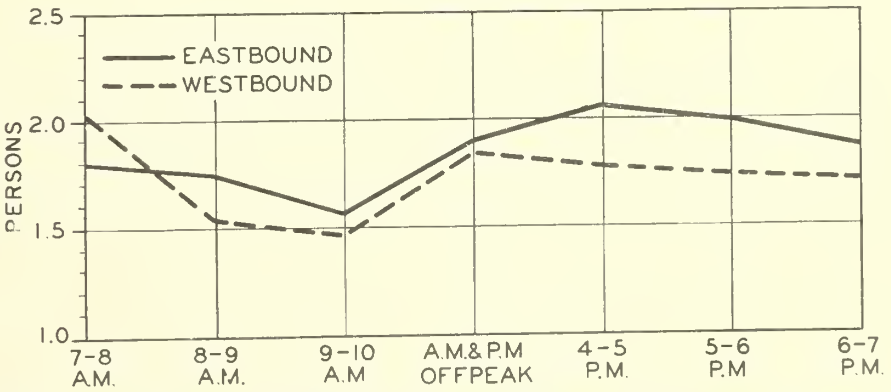

Figure 5.—Average weekday persons per auto during peak and offpeak periods for all three river crossings combined.

istic by application to known seasonal or annual data as follows:

$$X' = Np = N\frac{x}{n}$$

where X is the known total for the characteristic under consideration.

Appendix B

Reliability of Results

Two basic types of errors are encountered in all sample surveys, to a greater or lesser degree, which affect the reliability of the survey results. The first is sampling error, which arises from the fact that the population characteristics as pictured by the sample may not exactly coincide with the characteristics which would emerge if the population were sampled in its entirety. The second is nonsampling error, which arises primarily from errors in processing the survey results and from bias in the sample due to nonresponse.

Throughout the conduct of this survey, every effort was made to keep these errors to a minimum. Furthermore, at a number of periods throughout the survey, audits of certain results of the survey were undertaken in order to compare them with recorded information of total traffic volumes by direction, by type of day, by type of vehicle, and so forth. All of these comparisons revealed close agreement between the survey results and the recorded volumes.

Sampling error

The distinguishing feature of a sample based on the mathematical theory of probability is the fact that it makes possible the computation of a sampling error. This gives a measure of the degree of reliability that has been attained in any particular value derived from the survey. At a given level of confidence, 95 percent, for example, the sampling error is to be interpreted as a range of values about the sample estimate which includes the "true value" in the population 95 times out of 100.

Perhaps an example in the use of sampling error will clarify its meaning. During 1958, a total of 13,357 automobile trip interviews were made on weekdays at the Lincoln Tunnel. The survey indicated that 12.2 percent of these automobiles traveled between the Manhattan CBD and areas in northwestern New Jersey and upstate New York. The theory permitted the application of this sample proportion to the entire population which, in this case, was the 12.6 million automobiles that actually used the Lincoln Tunnel on weekdays in 1958 (50,343 on an average weekday). It is true, of course, that if the origin and destination patterns of every one of these 12.6 million movements via the Lincoln Tunnel on weekdays could have been ascertained, the proportion would probably be somewhat different from the 12.2 percent mentioned above. With a probability sample it is possible, for a given level of confidence, to estimate the maximum size of this difference by establishing a range of values about this sample estimate within which the true value should lie. This range can be expressed as follows:

$$Ps \pm t_{\alpha} Ps(C.V.)s$$

where Ps = proportion of some sub-class for characteristics s.
$(C.V.)s$ = coefficient of variation for characteristic s.
t_{α} = Student's t value for four degrees of freedom and α error.

The coefficient of variation measures the standard deviation relative to the mean.[2] Thus in this survey,

$$(C.V.)s = \sqrt{\frac{\Sigma(Nh_1 - Nh_2)^2}{[\Sigma(Nh_1 + Nh_2)]^2}}$$

where Nh_1 = card count of characteristic for each of the four seasons in subsample group 1.
Nh_2 = card count of characteristic for each of the four seasons in subsample group 2.

To make possible the use of the above formula for the computation of the coefficients of variation, the sample was drawn in two independent subsamples for each season.

In order to make sure that the sampling error computations did not overstate the reliability of the sample, a conservative formula was used which for most cases leads to estimates of error somewhat greater than actually exist for the ratio estimates developed in this survey. Thus, the results were probably a little better than the calculated errors would indicate.

In the example already given, $Ps = 12.2$ percent and $(C.V.)s = 0.034$. For a 68-percent level of confidence the t value is 1.14, and the true value in the population for the proportion of automobile trips on weekdays via the Lincoln Tunnel between the Manhattan CBD and northwestern New Jersey and upstate New York would be included in the range:

$$12.2 \pm 1.14 \times 12.2 \times 0.034 = 11.7 \text{ to } 12.7 \text{ percent}$$

For a 95-percent level of confidence the true value in the population lies in the range:

$$12.2 \pm 2.78 \times 12.2 \times 0.034 = 11.0 \text{ to } 13.4 \text{ percent}$$

Table 12.—Standard deviations for selected sample estimates

Sample size	Proportion of selected subclass in sample, *Ps*	Standard deviation, *Ps* (*C.V.*)*s*
	Percent	*Percent*
1, 353	75.7	1.5
918	44.2	2.2
2, 829	29.0	1.0
5, 462	22.0	.9
11, 745	14.3	.7
10, 355	12.7	.6
13, 357	12.2	.4
5, 707	9.5	.4
10, 355	6.7	.3
22, 444	4.3	.2
22, 444	3.2	.1
13, 357	1.8	.1
13, 357	1.7	.1
6, 836	1.7	.1

Table 12 shows the standard deviations f a series of representative estimates taken fro this report. There were literally over a m lion estimates obtained in the course of tab lating the survey results. The standar deviations shown are believed to be represent tive of the errors expected on any estimates this report.

Nonsampling error

In sample surveys involving the questioni of people, a certain amount of nonresponse bound to occur. The problem is to keep within such bounds that the theory of prob bility still is valid.

As mentioned previously, there were 57 primary sampling units (8-hour shifts) picke at random throughout the year, segregated in the four seasons of 1958. Of the 572 shif originally scheduled, 60, or 10 percent, we missed and had to be rescheduled to a ne date in the quickest possible time. All shif missed were due to the interviewer's sickne or some other personal reason.

The proportion of shifts rescheduled f each facility did not vary considerably fro the overall percentage (10 percent); hen there was no particular bias of reschedulin against any one facility. More importa than this, most shifts were rescheduled with a few weeks, and it is fair to assume that t general patterns of travel on the reschedul dates were not significantly different than the originally scheduled dates. Of cours on a few occasions, rain or snowstorms o curred on either the original or reschedul dates, thereby probably affecting the resul to some small degree.

It was only in the fourth season (Octobe December) that any considerable reschedulin lag occurred, but even here almost 70 perce of the shifts were completed within 4 weeks the original date. In the other three seaso at least half of the rescheduled shifts we completed within 1 week and all were co pleted within 4 weeks of the original dat

The number of interviews obtained rescheduled shifts were, on the whole, great than would have been taken on the origin survey dates, the average loads being 178 a 162 interviews, respectively. However, sin the procedure followed in this survey was reweight the number of the interviews c tained on the rescheduled dates to the numb which would have been obtained original this numerical discrepancy made no differenc The important assumption made was th the patterns of travel were not significant different on the rescheduled dates than the original ones. This seemed valid wh the time lag for rescheduling was as small that which occurred on the survey.

As in all sample surveys involving t questioning of people, there were some uni for which no information was obtained d to refusal of motorists to answer and other reasons. The latter, classified as no interviewable, included those sampling uni such as military vehicles, ambulances, a hearses which, although they were included the sampling universe, could not be inte iewed for obvious reasons. These two types f errors of nonresponse were treated by iking random selections from the obtained iterviews in an appropriate manner.

---

[2] Kish and Hess, *On Variances of Ratios and Their Differences in Multi-Stage Samples*, Journal of the American Statistical Association, June 1959.

The overall nonresponse rate experienced i this survey was only 1.5 percent of the otal number of interviews taken. There as no significant difference by facility or eason, the minimum being 1.0 and the maximum 2.9 percent. This is considered excellent for this type of survey and is an indication of the training in interviewing techniques which each interviewer received, as well as their patience and fortitude on the job.

Another error that occurs in surveys of this nature is due to the miscoding of the information obtained in the field. Procedures based on valid techniques of quality control were applied in the verification of coded interviews to insure acceptable standards of coding efficiency. Most of those coding errors that evaded detection and were key-punched incorrectly were discovered on checking the listing of all detail cards. Overall, processing errors of all types amounted to less than 0.2 percent.

Properties of Highway Asphalts—Part II, Various Penetration Grades

(Continued from page 85)

asier. Continued research is needed to re- ite the engineering properties of asphalts to hese fundamentals and to determine if these echniques are suitable for specification pur- oses.

Over the years attempts have been made to efine the properties of the asphalts in terms f their chemical composition or component nalysis, but often have resulted in uncertain sults because of the complexity of the or- anic molecules present. However, research ow being conducted in this area is encourag- ig (2, 3). New approaches with modern techniques may do much to provide fundamental information that ultimately may assist the researcher in solving the problems associated with the production and utilization of asphalts of high quality.

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New Publications

The Bureau of Public Roads has recently published two new bulletins. One of the bulletins, *Hydraulics of Bridge Waterways*, is the first of a proposed series on hydraulic design of highway drainage structures. The 53-page illustrated publication by the Division of Hydraulic Research is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 40 cents per copy.

Hydraulics of Bridge Waterways presents simplified methods for computing backwater caused by bridges, developed from extensive model tests and checked against actual measurements at bridge approaches. The empirical curves and methods of calculation contained in the new publication have been subjected to extensive field use during the past few years by State highway departments, consulting engineers, and the Bureau of Public Roads, and a number of improvements developed during this period are incorporated in the publication.

The nature of the new bulletin is indicated by the chapter titles: computation of backwater; extent of backwater; difference in level across approach embankments; dual bridges; abnormal stage-discharge condition; effect of scour on backwater; illustrative problems; and limitations of data.

As noted in the introduction to the publication, hydraulics should play an important role in establishing what the length and vertical clearance of a bridge should be, and where the bridge should be placed. Confining flood water unduly may well cause excessive backwater with resultant damage to upstream land and improvements and overtopping of the roadway, or may induce excessive scour endangering the bridge itself. Too long a bridge may cost far more than can be justified by the benefits obtained. Somewhere in between is the design that will be the most economical to the public over a long period of years, and finding that design is the objective of the bridge engineer.

The new Bureau of Public Roads publication *Hydraulics of Bridge Waterways* is intended to provide, within the limitations described, a means of computing the effect of a given bridge upon the flow of the stream it is proposed to span.

Classification of Motor Vehicles, 1956–57, reporting detailed information on the numbers of vehicles registered in 1956, with an estimate for 1957, classified according to type of vehicle, kind of fuel used, gross vehicle weight, and type of use—farm, nonfarm, for hire, and publicly owned—is also available from the Superintendent of Documents, U.S. Government Printing Office.

The 123-page bulletin contains a wealth of detailed information that has not been available since the truck and bus inventory undertaken in 1941 as a wartime preparedness measure. It represents a considerable effort on the parts of the States, the Bureau of Public Roads, and the bus industry, since the information in the detail compiled in this study is not available in the regular registration records of many States.

In addition to a brief, explanatory text and summary tables and illustrations, the publication contains three extensive series of tabulations which will be extremely useful to highway engineers, administrators, and econ mists, as well as to others interested in tran portation and government. The first of thes reports, by States, the numbers of vehicl classified by type, by class of use, and by kin of fuel used. The second series reports, als by States, the numbers of trucks and combin tions classified by types and by gross vehic weight groups. The third series reports, b Census divisions, the numbers of trucks an combinations classified by type, class of us kind of fuel used, and gross vehicle weight.

A fourth series of tables reports, by State the farm registration rates and limitations o farm registrations, in those States that have separate farm registration classification; an the numbers of trucks and combinatio registered in that classification, by type vehicle and gross vehicle weight.

One of the problems faced in the study wa the fact that the States use varying bases fo weight classification in their registration sys tems. To overcome this difficulty a series conversion tables were devised, for relatin known empty vehicle weight to probable gros vehicle weight. The conversion tables ar included in the publication, since they shoul be of considerable use to many who use moto vehicle statistics in their work.

Another problem faced was that of duplica tion of registrations. While this occurs bu little for most vehicle groups, it is a commo practice for commercial buses. Howeve duplicate bus registrations were eliminate through the cooperation of the bus industry

Classification of Motor Vehicles, 1956–5 may be purchased from the U.S. Governmen Printing Office, Washington 25, D.C., a 70 cents per copy.

U.S. GOVERNMENT PRINTING OFFICE: 1960

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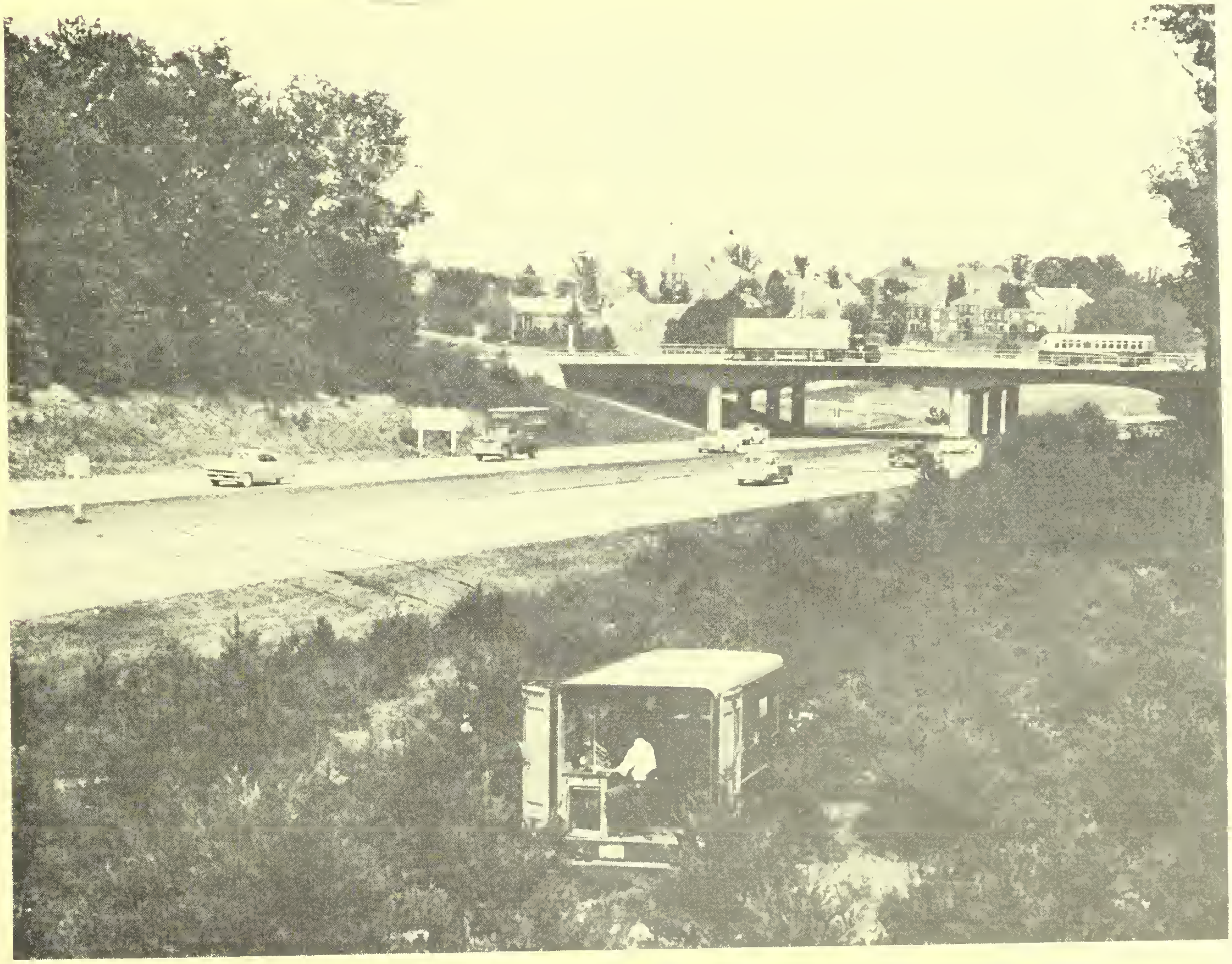

IN THIS ISSUE: The development and use of the traffic analyzer vehicle.

(Virginia Route 350, southwest of Washington, D.C.)

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Characteristics of Travel to a Regional Shopping Center

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by JACOB SILVER, Transportation Economist,
and WALTER G. HANSEN, Highway Research Engineer

This article describes many of the characteristics of travel associated with a shopping center which has a regional market area. The shopping center studied was located in a Virginia suburb of the Washington, D.C., metropolitan area.

Almost one-half of the trips generated by the shopping center for convenience-goods merchandise (food, drug, personal service, etc.) were made by residents living within 10 minutes driving time from the center. The average duration of parking for all convenience-goods shoppers was less than 50 minutes. Trips made to the center for the purpose of obtaining shopping-goods merchandise (apparel, furniture, etc.) were found to be greater in number, farther in distance, and longer in parking duration.

Only a limited degree of attraction to the center was due to its multiple-purchase characteristics. However, one-half of the weekly trips were made for apparel merchandise.

This article also points out the additional demand placed upon the arterial in the immediate vicinity of the center and how the additional traffic conflicts with normal peak-hour traffic movement. Also, an estimate of the impact of the shopping center upon other commercial complexes is discussed.

COMMERCIAL concentrations or "shopping centers" are continuing to be established in the increasingly populated suburban areas of our cities. The increasing traffic problems resulting from the change in the pattern and distribution of shopping travel to these new retail centers have become familiar to many highway and traffic engineers, as well as to metropolitan planners.

In 1955, a home-interview transportation survey—origin-destination (O–D)—was conducted in the Washington, D.C., metropolitan area. Shortly after the completion of this survey, a shopping center with a regional market area was opened in one of the Virginia suburbs. A special survey of travel to this center was undertaken for the purpose of updating the 1955 O–D survey data and, secondly, to provide information that would be useful in estimating the travel to similar types of centers that were expected to be built in the future.

This report is an analysis of the travel to this regional center. Although limited in scope, it presents some of the characteristics of travel that can be expected when a trip attractor of this nature is established. It should, therefore, be helpful to those responsible for the location, design, and operation of these centers and the highway facilities which serve them.

Summary

This article describes some of the characteristics of travel associated with a regional shopping center. It shows that this type of commercial development generates traffic at sufficient volumes during the normal peak hours of travel so as to create serious problems of traffic congestion on the arterials in the immediate vicinity of the center.

An analysis made of the trips for shopping goods and for convenience goods showed that for comparable floor area size, shopping-goods trips were generated at a rate significantly greater than the number of trips generated for convenience goods. Also significant were the differences in trip length and parking. Convenience-goods traffic generated shorter trips with shorter durations of parking than did shopping-goods traffic. From the data collected, it was found that convenience-goods traffic utilized parking space approximately twice as effectively than did the shopping-goods traffic.

Even though Seven Corners offered a large selection and variety of merchandise, which would permit multiple purchases at a single stop, the center only derived a limited degree of attraction to it due to this characteristic. The bulk of the trips attracted were for the purpose of purchasing shopping-goods merchandise, particularly apparel. This one group accounted for one-half of the total weekly trips.

Trips were attracted to the shopping center from throughout the area under study. The frequency of trips originating at a given distance or residential area, however, is dependent upon that center's location, composition, and size relative to these same factors of competing centers.

Seven Corners Shopping Center

Seven Corners shopping center, as shown in figure 1, is located at the eastern edge of Fairfax County, Va., at the intersection of several major highways. This location, approximately 7 miles, or 30 minutes driving time, from the District of Columbia central business area, was at the periphery of the urban development at the time of the study. The distribution of population in terms of driving time from the shopping center is shown in table 1. The *inboard* and *outboard* designations used in this table and subsequently refer to an arbitrary division of the market area into two parts, and are used only for comparative purposes to indicate that shopping characteristics were not uniform throughout the study area. The inboard area designates the area between the Seven Corners shopping center and the District of Columbia CBD; outboard designates the area from the shopping center away from the CBD. The dividing line is shown in figure 1.

The site of the shopping center is triangular in shape and, at the time of the study, encompassed a total area of 32 acres. The shopping center was bounded on the north by U.S. Route 50, a major east-west 4-lane highway, and on the south by Virginia State Route 7, a 4-lane, northwest-southeast highway. These two routes, together with other intersecting arterials in the vicinity of the center, make this location highly accessible for the residents of the Virginia section of the Washington, D.C., metropolitan area.

The site plan, illustrated in figure 2, indicates the location of the building structures, the parking areas, and the points of vehicle-access to the center. The main building is an I-shaped, 2-level structure with a department store at each end. The remaining establishments, with the exception of a supermarket and an automobile parts and service establishment, are located along a covered promenade (at both levels) between the two department stores.

The total floor area of the center, at the time of the study in 1957, was 600,000 square feet, of which approximately 503,000 was gross rental area. The types and number of estab-

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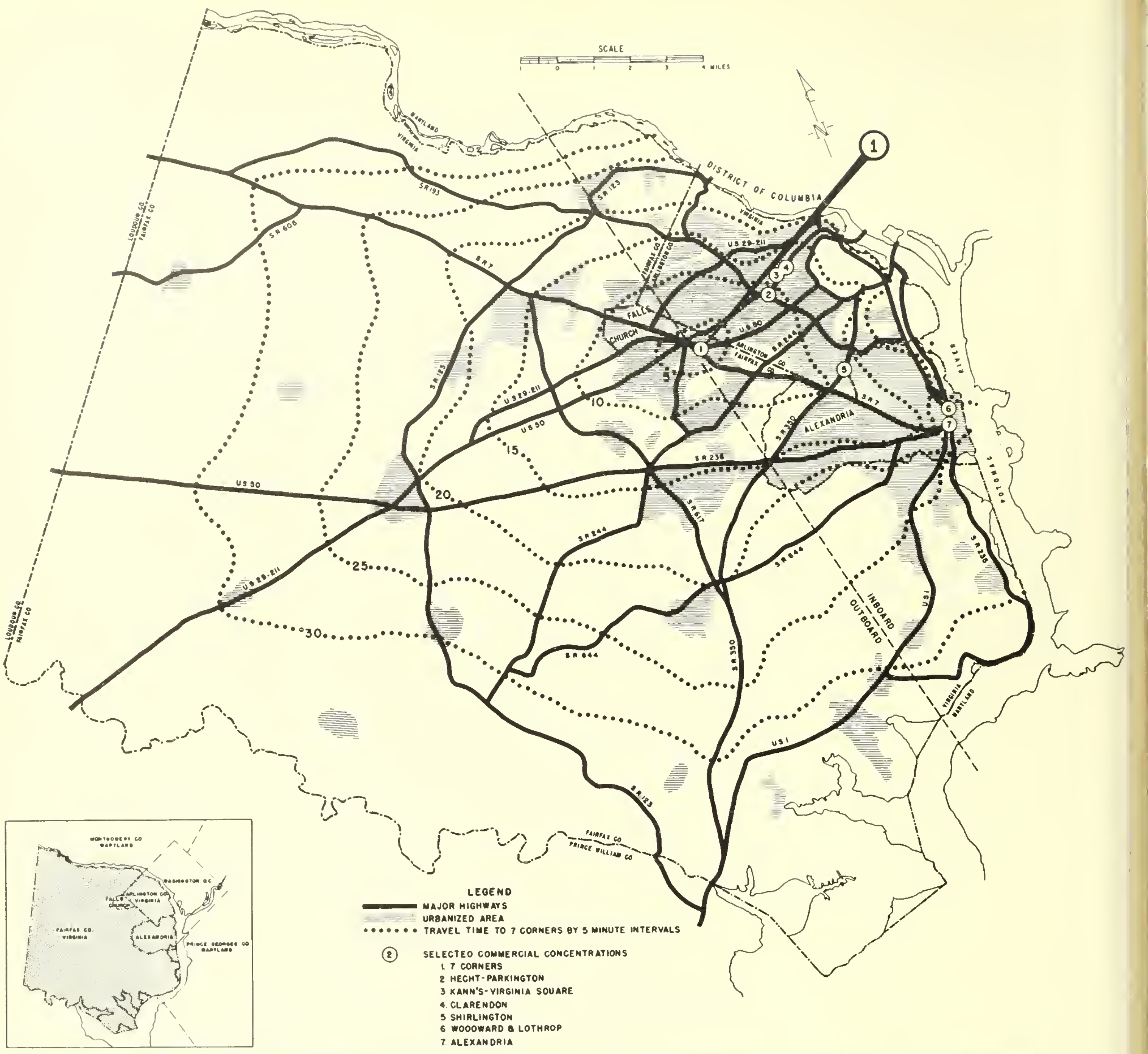

Figure 1.—Seven Corners shopping center and other commercial concentrations in the Virginia section of the Washington, D.C., metropolitan area.

lishments of the center, along with the space occupied by each category, are shown in table 2. Approximately one-half of the total gross rental area was used for the actual retailing of merchandise. About 1,000 persons were employed full or part time at the center.

Customer parking areas were convenient to both the upper and lower levels of the center, and provided space for 2,025 vehicles, or 4 spaces per 1,000 square feet of gross rental area. In addition, 235 parking spaces were provided for employees in an area opposite the shopping center, across U.S. Route 50.

The Travel Survey

The survey of travel to Seven Corners shopping center was conducted from April 8 through April 13, 1957, 6 months after the center was officially opened. Although 6 months was a relatively short period of time, it was considered sufficient for the people of the area to have become familiar with the new shopping and service opportunities and to eliminate possible bias created by sightseers to the new center.

Since Easter was on April 21, it was realized that the results would be affected somewhat by the holiday shopping. However, there is a considerable seasonal fluctuation in the purchase of many categories of merchandise and a survey made in any one week would not be representative of the entire year. In this case, the purchase of apparel was probably above average and this should be taken into consideration in appraising the results.

Information concerning the origin, lengt purpose, and duration of trips to the center, a well as automobile occupancy and vehicle reg istration, was obtained by personal interviews Every tenth parking space in the main park ing area was designated as an interview space Interviews were obtained on arrival for eac family represented by the occupants of th cars which parked in those interview spaces To be comparable with the 1955 travel survey children less than 5 years of age were not in cluded in the sample. In all, 1,558 interviews were obtained, representing about 7 percent o all the families arriving at the center during the interview periods.

Interviews were made on three days Wednesday, Thursday, and Saturday, an were expanded to represent weekly volumes

Table 1.—Distribution of population and the average income for each of several travel-time groups to Seven Corners shopping center

1957 travel time to Seven Corners (minutes)	For total area:[1]			Distribution of each travel-time group			
				Inboard		Outboard	
	Population [2]		Average family income [2]	Percent of population	Average family income	Percent of population	Average family income
	Number	Percent					
0–4	5, 600	1	$6, 500	68	$5, 800	32	$8, 500
5–9	54, 400	13	7, 800	57	8, 600	43	6, 600
10–14	129, 500	32	7, 500	65	8, 200	35	5, 900
15–19	73, 700	18	7, 900	89	8, 000	11	5, 800
20–24	73, 600	18	6, 900	61	7, 500	39	5, 800
25–29	32, 000	8	6, 400	74	7, 000	26	4, 400
30 and over	39, 300	10	5, 500	31	5, 800	69	5, 400
Total or average	408, 100	100	7, 100	65	7, 900	35	5, 800

[1] Only includes area in Virginia section of the Washington, D.C., metropolitan area.

[2] Unpublished data, Mass Transportation Survey, National Capital Planning Commission and National Capital Regional Council, 1955.

on the basis of directional traffic counts made throughout the week of the study at each of the eight access points shown in figure 2.

The interviews made on Wednesday were expanded on the basis of hourly traffic counts to represent trips made during the two days, Tuesday and Wednesday, when a majority of establishments were closed evenings. Interviews made on Thursday were expanded to represent trips made during Monday, Thursday, and Friday, when the entire center was open evenings. Saturday interviews were expanded to represent only Saturday trips. No attempt was made to adjust the data for any seasonal variations which might have existed.

No interviews were obtained from persons walking to the center or those arriving by bus. Pedestrian trips were omitted after the study indicated that the distances between the shopping center and most nearby residential developments, as well as the heavily traveled arterials surrounding the center, were not conducive to walking trips, and few walking trips were actually observed. Transit trips were omitted from the survey after sample counts showed an average of less than 150 persons per day arriving by bus.

In addition, no interviews were made in the area set aside for employee parking. Comparatively few employees actually used this space. Except for Saturday, less than 50 vehicles per day were parked there. Many employees parked in the customer parking area, even though there were regulations to the contrary. Also, observations showed that many of the employees were parking in the adjacent apartment development, while still others were brought to work by vehicles which did not park.

The work-trip data presented in the following sections are only for the center employees and noncenter workers, e.g., carpenters and delivery men, who parked their vehicles in the customer parking area. As a result, work trips are considered to be substantially underreported.

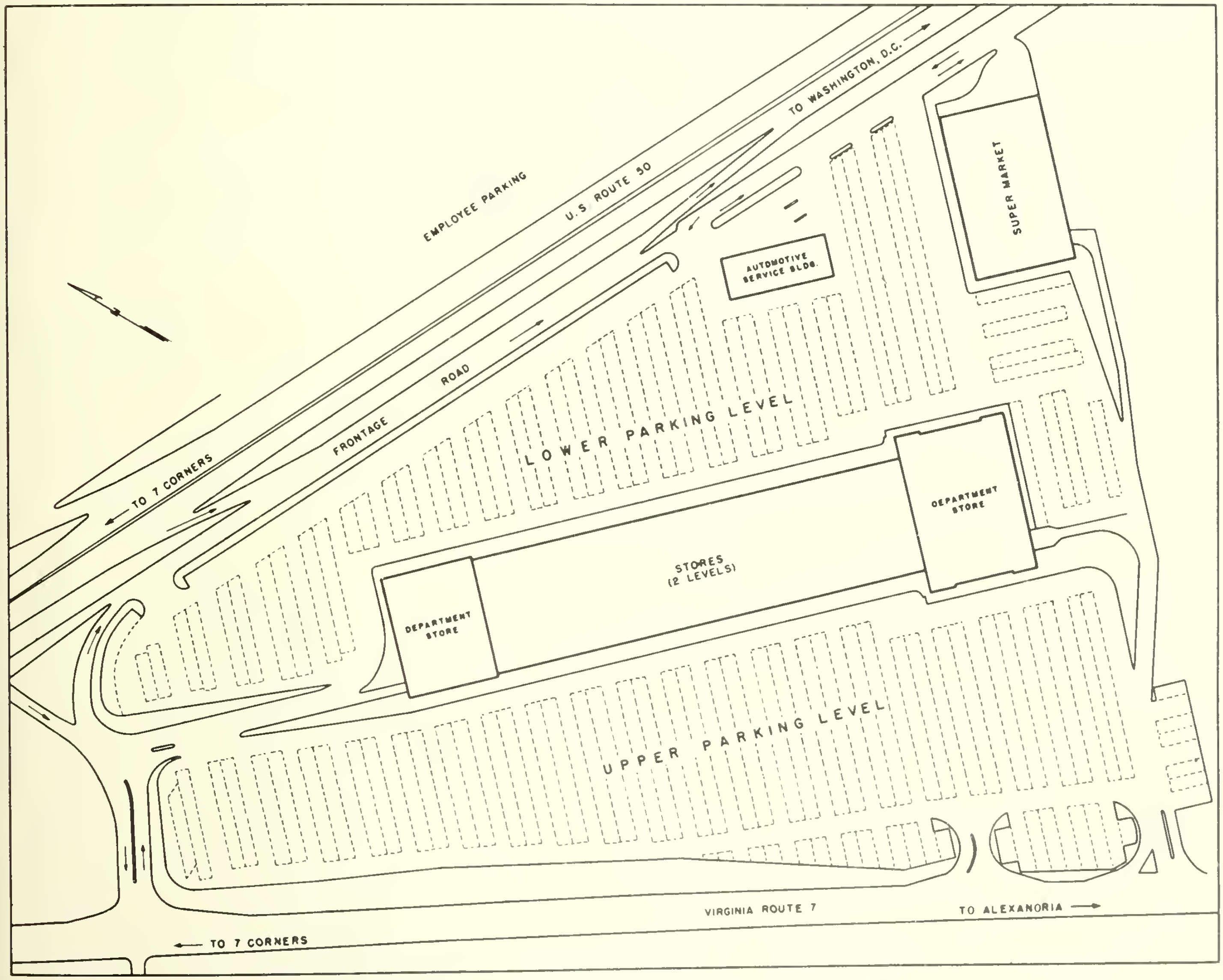

Figure 2.—Physical layout of Seven Corners shopping center.

Table 2.—Distribution of gross rental space at Seven Corners shopping center [1]

Category of business establishment	Number	Gross rental space: Area (square feet)	Gross rental space: Percent of total
Department store	2	210,700	42
Apparel	13	100,300	20
Food, drug, hardware, variety	7	96,000	19
All other shopping	11	50,500	10
Eating	1	15,900	3
Service	6	9,300	2
Occupied space	40	482,700	96
Unoccupied space	4	20,100	4
Gross rental space	44	502,800	100

[1] Data as of April 8, 1957.

Findings of the Study

During the week of the study, a total of 52,500 passenger vehicles, carrying an estimated 96,300 persons, entered the Seven Corners shopping center. These volumes were distributed among the days of the week as follows: Monday, Thursday, and Friday, 17 percent each; Tuesday and Wednesday, 12 percent each; and Saturday, 25 percent.

The volume of passenger vehicles entering the center by hour for each day of the week is shown in figure 3. Three distinct patterns were observed: One for those days when the center was open in the evening; one for those days when the majority of the establishments were closed in the evening; and one for Saturday. Of particular interest was the lack of any appreciable variation in the weekday volumes between the hours of 8 a.m. and 4 p.m. The fact that the center was open during an evening seems to have little or no effect on the volume of shopping during these daytime hours.

Peak traffic volumes

The peak period for traffic entering the center occurred on Saturday afternoon between noon and 5 p.m. The greatest volume during this sustained peak occurred between 2 and 3 p.m. when a total of 1,550 vehicles, or 3 percent of the total vehicles attracted during the week, arrived at the center. If the volume for this one hour were expressed as a proportion of the average daily weekday traffic (the usual base for reporting peak-hour traffic movement), it would be equal to 21 percent of the total number of motor vehicles arriving at the center during an average weekday, Monday through Friday. If both the arrival and departure of vehicles were considered, the peak hour of traffic occurred during the same hour with a total movement of 3,100 vehicles. This volume was also 3 percent of the weekly movement.

Although absolute peak volumes of traffic are important to the design of a center, particularly with respect to the design of access, the volumes which occur during the hours of peak-traffic flow on adjacent arterials are also of considerable importance; especially as it is this movement which will affect the operation of the highway proper. The normal Monday-through-Friday peak-hour flow on the highways in the Seven Corners area occurred between 5 and 6 p.m. The shopping center generated an average of 1,500 vehicle trips, arrivals plus departures, during this time period (the volume entering and exiting was evenly divided between the two adjacent arterials, Routes 50 and 7). Of the 1,500 vehicle trips, approximately 80 percent had originated at home and were not making intermediate stops as part of the work-to-home rush-hour movement. These 1,200 home-based vehicle trips give some indication of the additional demand placed upon the arterials in the immediate vicinity of the shopping center. If these volumes were not anticipated, this additional burden, conflicting with normal peak-hour traffic movement, would be large enough to create a serious problem of traffic congestion on the arterials serving this type of commercial development.

Parking facility

How much parking space should be provided is always one of the major problems in the design of a shopping center. As may be seen in figure 4, there were only 10 hours during the study week when the accumulation of vehicles at Seven Corners reached 50 percent or more of parking space capacity—2,025 spaces. On Saturday, the accumulation of vehicles surpassed the capacity of the parking area by 2 percent, resulting in considerable congestion within the parking area as well as at the entrances to the center.

Figure 4 also shows the percentage of parking spaces used for the two major categories of merchandise being purchased—shopping goods and convenience goods. *Shopping goods* were those which the customer, in the process of selection and purchase, characteristically compared the suitability, quality, price, and/or style of the goods; where the customer willingly made a special added effo in the purchase of these goods. For the pu poses of this study, trips for shopping goo included apparel, furniture, and special merchandise, such as flowers and jewelry.

Convenience goods were generally iten which the customer desired to purcha frequently, immediately, and with a minimu of effort. Habit played an important part the purchase of convenience goods. For tl purposes of this study, trips for this catego included food, drug, variety, and hardwa merchandise, as well as personal services, su as barber shop, laundry, and dry-clean establishments.

The rate at which vehicles arrive at tl center and the length of time that the remain are two factors in the determinati of the amount of parking space required at shopping center. The average duration parking by trip purpose at Seven Corners shown in figure 5. The most significant con parison to be made is the difference in tl duration of parking between trips to purcha shopping goods and those made to purcha convenience goods. Fifty-five percent of tl trips attracted for the purchase of shoppin goods parked 1 hour or less; the average dur tion of parking was 1.5 hours. In compariso 80 percent of the trips attracted for co venience-goods purchases parked for less tha 1 hour, with an overall average duration parking of 0.8 hours.[1] The above percentag mean that given equal and sustained rat of arrival, shopping-goods traffic, on th

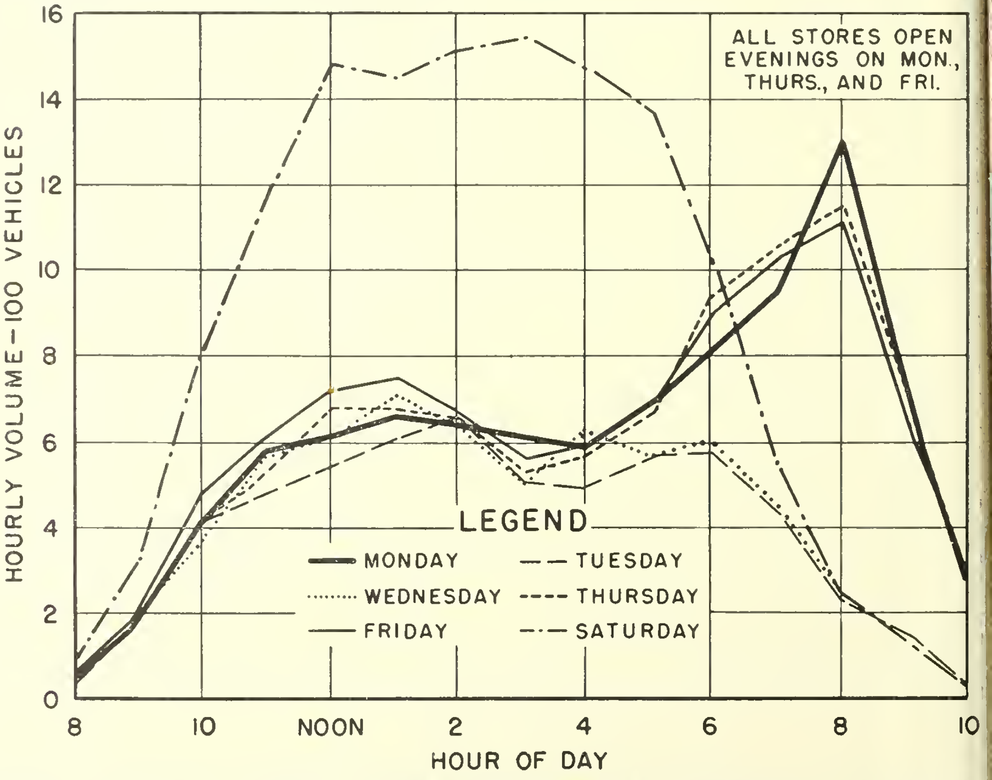

Figure 3.—Volume of vehicles arriving at Seven Corners shopping center by time of day and day of week.

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[1] The curve in figure 5 pertaining to trips for the purpo of eating is undoubtedly somewhat in error as evidenced b the large percentage, 25 percent, of trips with durations i excess of 1 hour. This error is felt to be due to an oversigh in the survey procedures which did not ascertain wheth or not people coming to eat at the center were also plannin to shop

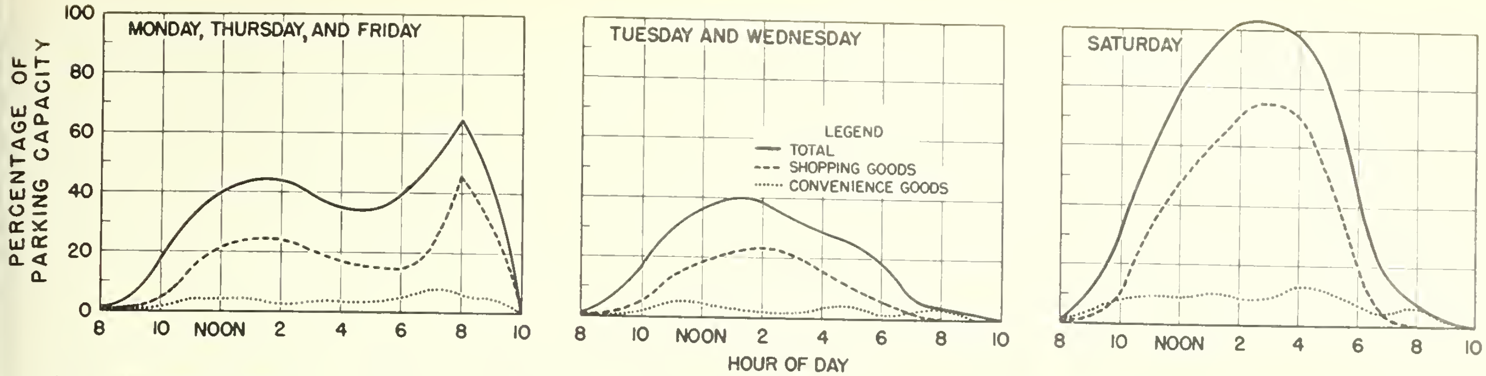

Figure 4.—Accumulation of vehicles at Seven Corners shopping center as a percentage of the 2,025-vehicle-space parking area.

average, required about twice as much parking space as did the convenience-goods traffic.

To provide a measure of the overall parking requirements for the traffic generated by these two classifications, the peak accumulation for each category, as shown in figure 4, was divided by the corresponding total weekly volumes of traffic. These measures, expressed in terms of parking spaces per 1,000 vehicles per week, were 48 spaces for shopping-goods traffic, and 26 spaces for convenience-goods traffic. For all purposes, 39 parking spaces per 1,000 vehicles a week were required.

Trip purpose distribution

The distribution of total trips arriving at Seven Corners shopping center by purpose of trip is summarized in table 3. Since the family is considered to be the basic consumer unit attracted to suburban shopping centers, the analysis of family trips has been emphasized in this report. For the purposes of this study, a family was considered to be either a single member of a family or two or more persons of the same family residing in the same dwelling unit, and family trips were measured by the number of families represented by persons arriving at the center.

As indicated in table 3, shopping-goods merchandise attracted slightly more than three times the volume of family trips than that which was attracted for convenience goods. The importance of apparel merchandise in attracting trips to the center is apparent because over one-half of the total travel to the center was made for this purpose.

The purpose distribution of all shopping trips made to the center, shown in table 3, indicated only the primary purpose for which a trip was made to the Seven Corners shopping center. Inasmuch as one of the stated advantages of a large regional center is to permit multipurpose shopping in a single stop, additional information was obtained to determine the extent to which people planned to shop for more than one category of merchandise before coming to the center. This is not to say that once the consumer began shopping at the center, other purposes were not considered; however, in the latter case, the secondary purchase was not considered by the individual upon leaving his place of origin.

Of the approximately 1,300 families interviewed who came to the center for the primary purpose of shopping, only 18 percent planned to shop for more than one type of merchandise at the time they arrived at the center. The number of primary trips and the percentage distribution of the secondary shopping trips are shown in table 4. In general, families who were primarily at the center to shop for convenience items and services were more likely to also be shopping for other categories of goods than were those primarily interested in shopping goods. This information would seem to indicate that although the regional shopping center offered a number of different types of merchandise which would permit multipurpose shopping at a single stop, the center attracted only a limited percentage of trips due to this factor.

Trip generation rates

The rates at which trips were attracted to Seven Corners are shown in table 5. There is a noticeable difference in the rate of attraction between the two major merchandise categories; the floor area devoted to shopping goods attracted family trips at a rate that was 20 percent greater than the floor area devoted to convenience goods. This differ-

Table 3.—Volume of trips to Seven Corners shopping center, by purpose

Purpose of trips	Weekly total of—					
	Vehicle trips		Family trips		Person trips	
	Number	Percent	Number	Percent	Number	Percent
Shopping goods:						
Apparel	27, 573	52	31, 322	53	53, 996	56
Other	2, 465	5	2, 586	4	4, 071	4
Unknown	1, 877	4	2, 407	4	3, 555	4
All shopping goods	31, 915	61	36, 315	61	61, 622	64
Convenience goods:						
Food	3, 768	7	3, 851	7	7, 376	8
Drug, hardware, variety	4, 337	8	4, 628	8	7, 373	8
Services	1, 836	4	1, 856	3	2, 520	2
All convenience goods	9, 941	19	10, 335	18	17, 269	18
Total shopping	41, 856	80	46, 650	79	78, 891	82
Work	4, 248	8	4, 849	8	5, 301	6
Personal business	581	1	670	1	877	1
Eat	4, 829	9	6, 161	10	9, 773	10
Serve passenger	968	2	968	2	1, 426	1
Total trips	52, 482	100	59, 298	100	96, 268	100

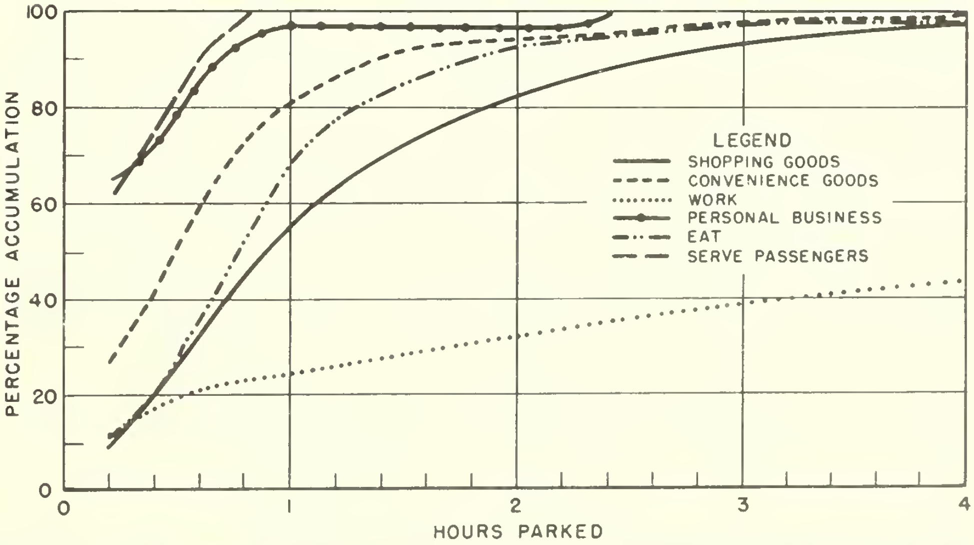

Figure 5.—The duration of time vehicles were parked at Seven Corners shopping center, by trip purpose.

Table 4.—Distribution of multiple-purpose family shopping trips

Primary purpose	Total	Percentage with no secondary purpose	Percentage of trips with secondary purpose				
			Shopping goods		Convenience goods		
			Apparel	Other	Food	Drugs, hardware, variety	Services
Shopping goods:							
Apparel	31, 322	84	------	4	4	7	1
Other	2, 586	90	3	------	1	4	2
Convenience goods:							
Food	3, 851	54	17	8	------	18	3
Drugs, hardware, variety	4, 628	94	3	3	------	------	------
Services	1, 856	60	26	11	------	3	------
Unknown	2, 407	------	------	------	------	------	------
Total or average	46, 650	82	3	4	3	7	1

Table 5.—Distribution of sales floor area and the trip-generation rates, by type of merchandise

Classification	Sales floor or dining area (square feet)	Percentage of floor area	Weekly volume of trips per 1,000 square feet of sales floor area		
			Vehicle trips	Family trips	Person trips
Shopping goods:					
Apparel	132, 100	52	209	237	409
Furniture	16, 000	6	43	45	71
Specialty items [1]	41, 400	16			
Total or average	189, 500	74	168	192	325
Convenience goods:					
Food	21, 700	9	174	178	340
Drugs	13, 900	5	111	119	189
Hardware	9, 300	4			
Variety	14, 800	6			
Services	5, 800	2	317	320	435
Total or average	65, 500	26	152	158	264
Total merchandise	255, 000	100	164	183	309
Restaurant and lunch counters	10, 000	--------	483	616	977
Gross rental area [2]	--------	--------	104	118	192

[1] Specialty items included jewelry, books, records, etc.
[2] Based on total gross rental area of 502,800 square feet.

ence in rate of attraction, however, is somewhat less when considered in terms of vehicles since persons purchasing shopping goods arrive, on the average, with slightly more than one family per vehicle.

The aforementioned rates of attraction have been expressed in terms of total weekly traffic. If they were to be expressed in terms of average weekday traffic, they would be equal to approximately 15 percent of the values shown in table 5.

Geographic Distribution of Trips

The spatial distribution of persons making trips to a regional center is important not only to the location and operation of a center, but also to the design and operation of adjacent and access arterials since the distribution of the market area helps determine the major directional movement of traffic to the center. In this article, only the results of trips to the Seven Corners shopping center by residents of the Virginia section of the Washington, D.C., metropolitan area [2] are analyzed. These trips accounted for approximately 89 percent of the total travel to the center. The other 11 percent of travel, derived from the District of Columbia, Montgomery and Prince Georges Counties, Md. (6 percent), and all other areas outside those already noted (5 percent), are not included in the following discussion.

The accumulated distribution of trips by travel time for residents of the Virginia section is compared with the distribution dwelling units in the same area in figure The more rapid increase in the curve of a trip purposes, as compared to the curve dwelling units, indicated that a greater pe centage or frequency of trips were made b residents living close to the shopping cente

There was found to be an even distributio in the actual number of trips between th areas designated as being inboard and ou board from the center. However, as may b seen in figure 7, if trip frequency is compare rather than total volumes, there is a significan difference in the distribution of trips. I general, the trip frequency to Seven Corner by families residing in the outboard area, wa greater than those made by families residin in the inboard area.

In figure 7, the extreme fluctuation in th 3 curves representing trip frequency from the outboard area between 15 and 30 minut is believed to be a result of the procedure use in analyzing the data. In summarizing tri and population information by distanc zones of the 1955 O–D traffic study were use As a result, it was necessary to arbitraril assign total zone figures to a single incremen of distance even though zones in this genera area were large and may have overlappe several time-distance bands.

From the more densely populated inboar area, where many of the competing commercia concentrations are located, including th Washington, D.C., central business district there was an average attraction of 310 weekl family trips per 1,000 dwelling units. From the outboard area, which had no major com peting centers, the average attraction wa 556 weekly family trips per 1,000 dwellin units. This difference in trips indicates th marked effect that competing shopping cen ters have on the rate at which trips will b attracted to a shopping center.

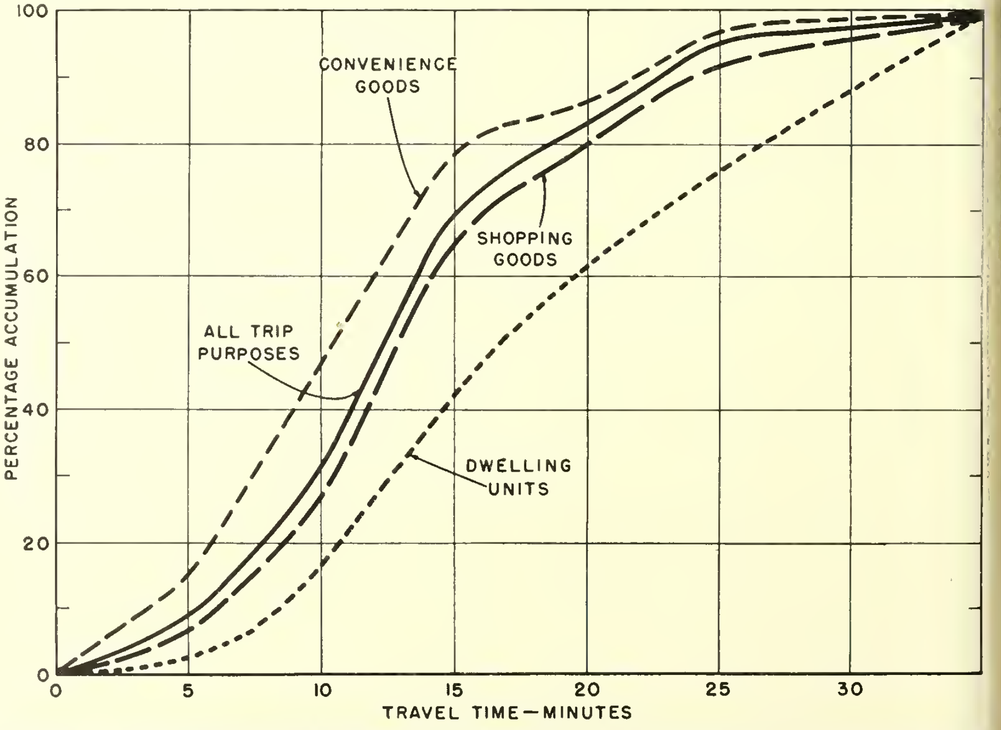

Figure 6.—Accumulated distribution of dwelling units and family trips made to Seven Corners shopping center, by travel time and trip purpose.

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[2] The Virginia section of the metropolitan complex included all of Arlington and Fairfax Counties and the cities of Alexandria and Falls Church. This area had a total population of 408,000 in 1955.

Although the frequency at which trips were attracted for both shopping and convenience goods decreased as driving time to the center increased, the rate of this decrease, as illustrated in figure 7, was less rapid for shopping-goods trips than for convenience-goods trips. The relative shortness of trips for convenience goods is further emphasized by the information illustrated in figure 6; almost one-half of the trips for convenience goods were made by families residing within 10 minutes driving time of the center. These families accounted for only one-fourth of the trips for shopping-goods purchases. Shopping-goods trips, on the average, tended to be approximately 25 percent longer than the convenience-goods trips.

Impact on Other Commercial Concentrations

The estimated impact of the Seven Corners shopping center on trips to other commercial concentrations is shown in figure 8. These estimates are based on shopping travel data obtained from the 1955 Washington metropolitan area transportation survey and the assumptions that the frequency with which people shop (i.e., trips per weekday per dwelling unit) would remain constant between 1955 and 1957, and that the relative attraction of trips to the selected commercial concentrations other than Seven Corners would remain the same in 1957 as it was in 1955.

Based on the above assumptions, the total number of shopping trips made by residents in 1957 within the Virginia section of the 1955 transportation study cordon, was obtained by multiplying the estimated 1957 dwelling units by the 1955 frequency of shopping trips per dwelling unit. After these totals were reduced by the average daily number of trips attracted to Seven Corners, the remainder was distributed to the other commercial centers in the same proportion as they were in 1955.

The three major commercial groups that compete with Seven Corners shopping center for shopping-goods purchasers were then selected for a comparison of shopping trips before and after the establishment of the center. Due to their close proximity and the relative ease of traveling from one to the other, the data for the three separate commercial concentrations in the Clarendon complex and the two separate concentrations in the Alexandria complex were combined into single groups. The third major commercial group was the Shirlington shopping center.

At the time of the study, there was no department store at the Shirlington shopping center. The Alexandria complex had two department stores, both limited in size, while the Clarendon complex had three department stores of 120,000 square feet or more.

Within the limitations imposed by the methods of estimation, the results illustrated in figure 8 give an indication of the changes in the directional movement of shopping travel that have resulted from the establishment of the Seven Corners shopping center. The major point indicated is not that the competing centers attracted fewer trips as a result of Seven Corners, for these losses will be offset for the most part by future population increases in the metropolitan Virginia area; but rather, it points up the fact that the frequency and volume of trips attracted to a given shopping center from a particular distance or residential area are dependent upon the location, composition, and size of that shopping center, relative to these same factors for other competing centers.

As this study was limited to an examination of travel to Seven Corners only, it was not possible to measure these "relative" characteristics with other commercial centers. Therefore, the relationships shown here are only indicative of the magnitude of the effect that Seven Corners had on the shopping habits and travel patterns of the area under study and are not directly applicable to a shopping center in a different setting.

The location of the four commercial concentrations and the percentage distribution of shopping trips estimated for each concentration is shown in figure 8. The areas, in which in 1957 each concentration attracted a greater percentage of shopping trips than did the other concentrations, are numbered I through IV to correspond to the Arabic numbers locating each concentration. Also shown is a comparison of domination for the years 1955 and 1957, before and after construction of the Seven Corners center. It may be seen that except for two areas, designated IA and IIA, each commercial concentration was dominant in the general area where the concentration was located. For example, the Clarendon complex was dominant throughout Arlington County except for the southwestern and extreme western sections where Shirlington and Seven Corners were located. It was difficult to explain the existence of the two exceptions, IA (where Seven Corners was dominant) and IIA (where the Clarendon complex was dominant), although there were several possible explanations. One explanation was that each area was located on or adjacent to a direct route to the centers of dominance. Another possible explanation advanced was that there was a special attraction to particular stores, especially department stores, within these centers as a result of such factors as charge accounts, quality, price, or type of merchandise, etc. (Fig. 8 is shown on following page.)

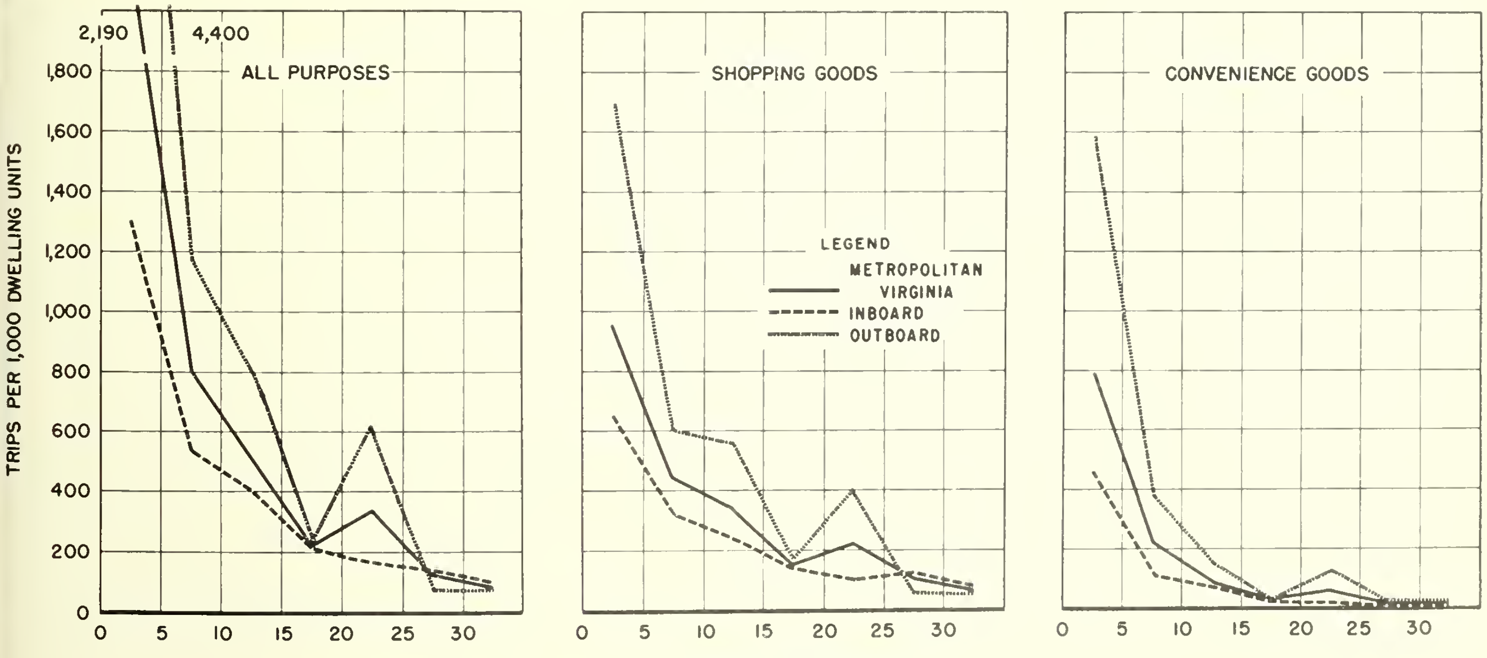
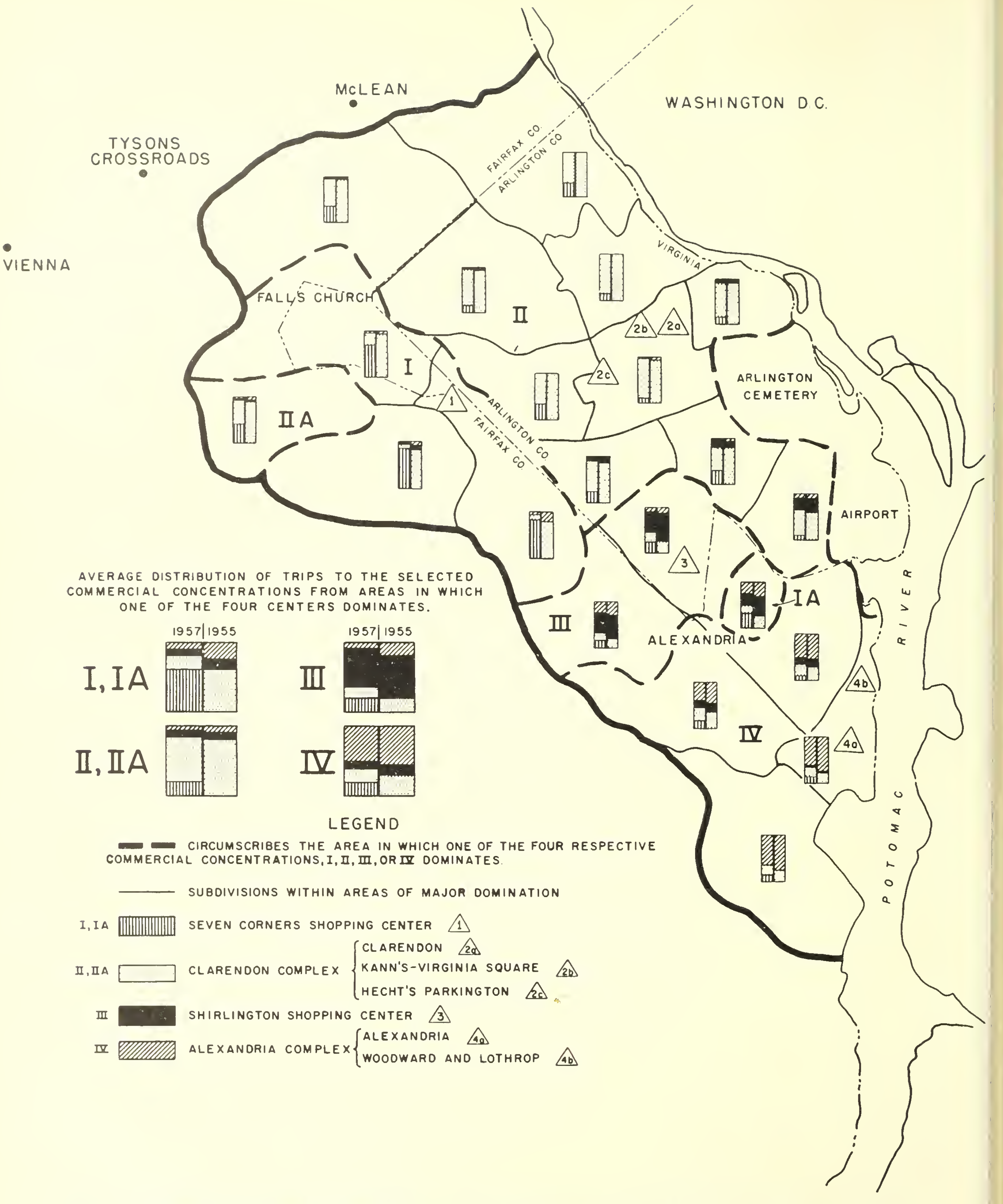


Figure 7.—Family trips per 1,000 dwelling units to Seven Corners shopping center, by travel time and trip purpose.


McLEAN
TYSONS CROSSROADS
VIENNA
WASHINGTON D.C.
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I
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2a
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ARLINGTON CO.
FAIRFAX CO.
ARLINGTON CEMETERY
AIRPORT
ALEXANDRIA
POTOMAC RIVER
AVERAGE DISTRIBUTION OF TRIPS TO THE SELECTED COMMERCIAL CONCENTRATIONS FROM AREAS IN WHICH ONE OF THE FOUR CENTERS DOMINATES.
1957 | 1955
I, IA
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LEGEND
CIRCUMSCRIBES THE AREA IN WHICH ONE OF THE FOUR RESPECTIVE COMMERCIAL CONCENTRATIONS, I, II, III, OR IV DOMINATES.
SUBDIVISIONS WITHIN AREAS OF MAJOR DOMINATION
I, IA SEVEN CORNERS SHOPPING CENTER 1
II, IIA CLARENDON COMPLEX: CLARENDON 2a; KANN'S-VIRGINIA SQUARE 2b; HECHT'S PARKINGTON 2c
III SHIRLINGTON SHOPPING CENTER 3
IV ALEXANDRIA COMPLEX: ALEXANDRIA 4a; WOODWARD AND LOTHROP 4b

Figure 8.—Percentage distribution of estimated shopping trips to selected commercial concentrations in 1955 and 1957.

The Effects of Enforcement on Traffic Behavior

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by RICHARD M. MICHAELS,
Research Psychologist

A carefully controlled study was conducted on the effect of different amounts of highway police patrol on accidents, traffic diversion, and speed. Analysis of the data indicated that although there was no significant change between test and control routes in average speed or percentage over the speed limit there was a significant reduction in the variance of speeds which was directly related to the enforcement level. However, no effects were found on diversion or accidents. The latter finding was at variance with the original report of the study and was due to the nature of the assumption about the accident distributions. In general, the results of the study indicated that the effects of enforcement may be very subtle and may influence traffic behavior in a very indirect fashion.

Finally, this article demonstrates the difficulties involved in the conduct of many kinds of field research. The inability to obtain adequate control over the experimental variables and the limited kind of measures available in such research often make interpretation of results difficult and generalization often suspect.

IT IS GENERALLY BELIEVED that police patrol significantly influences accident incidence, especially where the patrol is employed selectively. In addition, there is some evidence to indicate that enforcement techniques affect the speed distribution of the traffic in its area (*1*).[1] However, there has been no clear-cut proof in support of either of these effects of enforcement, that is, on accidents or speed. Furthermore, there are no data to demonstrate how the amount of patrolling is related to changes in traffic behavior.

The purpose of the study reported here was to examine the effects of different degrees of enforcement on three traffic variables. First, it was hypothesized that the speed distribution of traffic should be affected in direct proportion to the amount of enforcement. Three specific effects or combinations of them are possible: (1) the mean speeds are significantly reduced; (2) the percentage of drivers exceeding the speed limit is significantly reduced; and (3) the variance in speed is significantly reduced. Second, if police patrol tends to inhibit drivers, they may then take alternate routes where available. Thus, it was hypothesized that traffic volumes on the patrolled routes should decrease while those on the parallel control routes should increase. Again, the change in volume should be directly related to the amount of enforcement. Third, if patrolling influences accidents, then it was hypothesized that accident incidence on the test routes should be reduced in direct relation to the amount of enforcement.

The experimental design and the data in this study were developed by the Traffic Institute of Northwestern University as part of a cooperative project with the Bureau of Public Roads and the Wisconsin State Highway Patrol. The work was originally presented in a report by R. P. Shumate (*2*).[2]

The present study is a re-evaluation of that data and arose in part out of a difference of opinion over the statistical assumptions employed in the original analysis of the accident data. The statistical assumptions employed here lead to different conclusions from the original report. The fact that there were differences indicates that conclusions may be highly sensitive to the statistical analysis employed, and that extreme care must be exercised in the choice of method.

A second and even more important reason for extended analyses of these data concerns the design of the experiment. It is believed that this study was one of the best designed studies that has been conducted in the enforcement area. It was actually because of this belief that a further analysis was worthwhile, for it allowed an insight into certain aspects of enforcement as well as the problems inherent in the conduct of this type of research. It should be clear, therefore, that the differences between the two reports are due to differences in the analysis and interpretation of the data and not to the design and conduct of the research.

Summary

The present study was an examination of the effect of differing levels of enforcement on three traffic variables: accidents, speed, and volume. It was hypothesized that as the level of enforcement was increased, accident frequency would go down; traffic would divert to alternate routes; and the mean and variance of the speed distributions would decrease. Measurements were made on four test routes in different parts of the State of Wisconsin. In three of the four cases a parallel control route was used.

The results indicated that the first two hypotheses were not supported. No reliable reduction in accidents occurred on any of the test routes when compared with their controls. No significant diversion of traffic occurred using deviation from volume trend as a measure of change. For the speed distributions, a decrease in mean speed was observed but was of the same order of magnitude as on the control routes. The variance of these distributions did, however, decrease for three of the four test routes while they did not for the control routes. The observed decreases were statistically significant. In addition, the decreases gave some indication of being directly related to the level of enforcement.

It was suggested that the results indicated that the effects of enforcement on traffic behavior were, at the very least, indirect. The enforcement had a relatively small effect on all the variables measured. The reasons for this were felt to be the crudeness of the measures employed and the unreliability of measurements inherent in any field study of this magnitude.

Procedure

In October 1955, study of the distribution of accidents throughout the State of Wisconsin was begun to select four routes on which to conduct tests. The principal criterion for selection of the test routes was frequency of accidents. In order to have any possibility of demonstrating some effects on accidents by enforcement, it was necessary to employ sections having a large number of accidents, since the reliability of any accident reduction requires a reasonably large sample of accidents.

Before January 1956, four test routes were selected. Each had an expected accident frequency high enough to provide an adequate sample. These routes, as shown in figure 1, are:

Test route No. 1—26 miles of U.S. Route 41, a 4-lane, divided highway from the Illinois State line to the Milwaukee County line.

Test route No. 2—52 miles of U.S. Routes 12, 18, and 51 in Dane County of which approximately 4 miles are 4-lane, divided highway with partial control of access and the remainder 2-lane highway.

Test route No. 3—104 miles of U.S. Route 41, a 2-lane highway from the north Milwaukee County line to Green Bay. Parts of this section had to be eliminated from the study because of major highway construction.

Test route No. 4—208 miles of U.S. Route 12, a 2-lane highway from Baraboo to the Minnesota State line.

To insure that data from the test routes did not merely reflect a general decrease or increase in accidents throughout the area, a series of control routes were established (fig. 1). Each control route had approximately the same length and the same design standards as the test route for which it acted as a control, but was geographically removed from the test sites and was not subjected to any enforcement increase. Each control route was evaluated in the same manner as the test routes. Such a control route was available for three of the four test routes.

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[1] Italic numbers in parentheses refer to a list of references on page 124.

[2] For ease in presentation, the study procedure as described in Mr. Shumate's report has been incorporated in this article.

Method of measuring enforcement

To make meaningful comparions between test routes it was necessary to have a measure of intensity of enforcement. Common methods of measuring enforcement effort use the number of recorded enforcement actions taken by officers in a specified time and area, such as arrest, citations, or warnings. Such methods were considered and rejected because probability of violation and subsequent detection will vary considerably from one test route to another. The method finally adopted is based on the probable frequency with which a driver traveling a section of highway could theoretically encounter a readily identified patrol vehicle, either stopped or in motion. It is expressed as the average number of patrol units a driver would pass per mile of travel. Thus,

$$\bar{M}=\frac{R}{M}$$

where:

$\bar{M}$ = The average number of miles driven for each patrol unit observed.

M = Miles of highway in the route under consideration.

R = The number of patrol units assigned to the route.

To study the effect of various intensities of patrolling, as defined in the preceding paragraph, it was necessary to vary either the number of enforcement units assigned to each test section or the linear miles of highway. The difficulty of finding four short sections of highway which had a sufficiently large number of accidents prompted the decision to vary miles of highway under enforcement rather than varying the number of enforcement units and keeping the length of test section constant. The variable highway mileage was doubled for each test route while the patrol units were maintained at eight for each test route.

Methods of assignment

Methods of assignment within the test routes were standardized. Patrol units assigned to test routes were relieved of all duties which would take them away from the test route. In those instances where assigned personnel were required to be away from the test routes for extended periods relief units were substituted.

To distribute patrol evenly over the test route, enforcement units were assigned to route sections ranging from 3 miles in test route No. 1 to 25 miles in test route No. 4. Enforcement units were required to work within their assigned sections except for emergencies or when in pursuit of violators. All enforcement units were readily identifiable as such, being marked by colored decals, two-tone color schemes, special number plates, roof lights, and police radio antennas. Occasionally unmarked units were used, but so seldom that any special effect they may have had would be negligible. No attempt was made to standardize enforcement techniques. Personnel had been taught several methods and were permitted to use whatever method seemed most appropriate at the time.

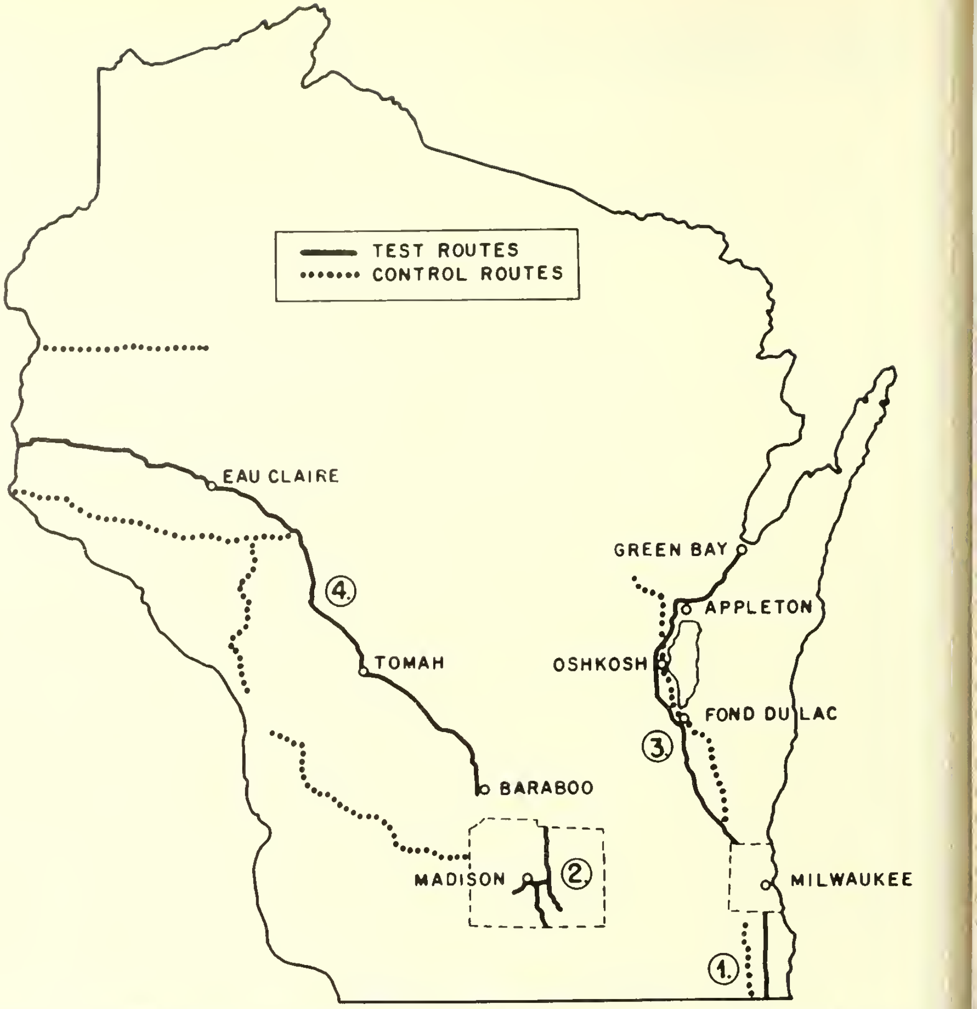

Figure 1.—Highway test routes and control routes used in Wisconsin enforcement study

In the interest of economy it was decided to maintain enforcement units on duty for only 18 hours each day. Study of accident patterns revealed that accidents during the 18 hours of highest frequency each day exceeded 80 percent of the total number of accidents occurring in any given period. The peak 18 hours for each test area was predetermined and assignments made accordingly. Work assignments were varied for different days of the week to accommodate differences in the daily accident pattern.

In January 1956, the first group of 2 enforcement officers were assigned to test route No. 1. Enforcement units, each consisting of a patrol car operated by a single uniformed officer, were assigned so that eigh units were on duty for each of the 18 hours o highest accident frequency. This required 1 men daily. The remaining eight men were used as relief for days off, sickness, an vacations. This method of assigning personnel was followed on each of the subseque test routes.

In April 1956, test route No. 2 was place under patrol using methods previously described. In August and November of the san year test routes Nos. 3 and 4, respectively were placed under police patrol. Table summarizes pertinent data concerning eac test route.

Accident data

Accident experience for each of the tes routes was compiled by year from 1950 throug

Table 1.—Assignment of patrol units by test route

Route No.	Patrol began	Duration of patrol, months	Miles of highway	Men assigned	Average units on duty (18 hours)	Patrol units passed by average vehicle per mile of driving
1	January 1956	24	26	24	8	0. 307
2	April 1956	18	52	24	8	. 154
3	August 1956	17	104	24	8	. 077
4	November 1956	13	208	24	8	. 038

Table 2.—Accident data on the experimental routes for years 1950–57

Year	Number of accidents on route No. 1				Number of accidents on route No. 2 [1]		Number of accidents on route No. 3				Number of accidents on route No. 4			
	Test route		Control route		Test route		Test route		Control route		Test route		Control route	
	Fatal and nonfatal injury	Property-damage-only	Fatal and nonfatal injury	Property-damage-only	Fatal and nonfatal injury	Property-damage-only	Fatal and nonfatal injury	Property-damage-only	Fatal and nonfatal injury	Property-damage-only	Fatal and nonfatal injury	Property-damage-only	Fatal and nonfatal injury	Property-damage-only
1950	88	92	24	26	83	108	58	84	27	41	223	329	86	164
1951	83	143	8	34	108	165	60	112	31	67	197	315	87	178
1952	100	136	14	15	102	187	65	78	56	47	219	388	126	188
1953	97	219	16	28	101	201	65	87	29	54	221	377	95	169
1954	109	170	15	24	97	199	50	64	27	48	131	327	98	171
1955	94	154	24	21	144	200	69	85	29	40	184	327	92	167
1956	78	190	17	31	114	187	81	91	33	37	189	315	97	171
1957	72	176	18	22	102	167	52	56	36	47	170	284	97	144

[1] No control route.

1957. These data were obtained from spot maps maintained by the State Highway Commission. Since Wisconsin law required every driver involved in an accident which exceeded $100.00 in total damage, or where a physical injury was sustained, to submit a report to the State Motor Vehicle Department, information on the spot maps was compiled from the accident reports.

Accident data included accidents occurring throughout all 24 hours of the day. Lack of complete records made it impossible to separate accidents occurring only during the 18 hours each day that enforcement units were on duty. However, because on the average 80 percent of all accidents occurred during the 18 hours of increased enforcement effort, it was assumed that changes in this large a portion of the total would not influence the magnitude of the entire sample.

Traffic volume data on each test section were obtained by years from 1951 through 1957 from the State Highway Commission. Samples of vehicle speeds at selected locations within the test areas were also obtained for periods of time both before and after the beginning of patrol operations.

Application of the chi-square test

A different statistical analysis was used for each hypothesis tested. For the accident data, in all but one case, a 2×2 chi-square test was employed to compare test and control routes for the 2 years, 1955 and 1957. In the single exception, route No. 2, where there was no control route, a simple chi-square was computed for the test route over the same 2-year time period. Thus, the chi-square test, in utilizing only 2 years of data—the before and after time period—disregards all other historical data that were available.

In the original report of this study, the assumption was made that there was a positive trend in accidents over time. Therefore, a trend line was fitted to the accident frequency, covering a 9-year period for which accident data were available. On this basis it was possible to predict what the accident frequency should have been in subsequent years if there had been no enforcement. By measuring the deviation of the accident frequency for the period of increased enforcement from that predicted by the regression line, it was inferred that a significant reduction in accidents occurred on the experimental routes. The existence of a trend was an extremely strong assumption, and it could not be wholly validated from the data. If, in fact, there was no reliable trend, then use of a trend equation with a positive slope would tend to insure a significant reduction in post-enforcement years.

Use of the chi-square test does not require such strong assumptions. On the contrary, the chi-square is a conservative test which used the control route data as the basic determinant of any significant changes in accident frequency on the experimental route. The test assumed no change in accident frequency from before to after, or that any changes in the test routes were due to factors extraneous to the enforcement variable which may have been expected to influence the control route equally.

The volume data were analyzed by measuring the deviation from the volume trends for the years 1950–56. Thus, a trend line was fitted to the data for these years. Changes occurring in volume in 1956 and 1957 were determined and compared with the volume expected on the basis of the trend line. The assumption that traffic volume is increasing with time is a reasonable one on the basis of national data (*3*).

The speed data were analyzed using the analysis of variance. The mean and standard deviation of the speed distribution on each route were computed for both the before year (1955) and the after year (1957). It was possible then to make comparisons both within and between each route.

Results of Analysis

The accident data for the test and control routes for the years 1950–1957 are shown in table 2. Analysis of the before and after data are shown in table 3. On all routes except No. 2, a 2×2 table was available and a chi-square test performed.

In the chi-square tests, the .05 level of significance was used. That is, the hypothesis that there was no reduction in accidents was rejected only if the measured decrease in accidents could have occurred by chance 5 or less times in 100. As may be seen, only in the case of test route No. 2, having 0.154 patrol units passed per mile, was there a significant reduction in fatal or injury accidents. In this one case, no control route was available so that a simple comparison was made between 1955 and 1957. It should be noted from table 2 that 1955 was an abnormally high year for accidents on this route. Consequently, ascribing a reduction in accidents to the increased patrol was probably unwarranted. Comparison of the accident data on the test routes with those of the rest of the State and with all the control routes are also shown in table 3. As may be seen, a significant reduction in number of accidents, both for test and control routes, occurred for all conditions except for property-damage-only accidents where the combined test and control data were used.

Table 3.—Effects of enforcement level on accidents, by test and control routes

Route No.	Patrol units passed per mile on test route	Year	Number of fatal and nonfatal injury accidents			Number of property-damage-only accidents		
			Test route	Control route	Significance of change [1]	Test route	Control route	Significance of change [1]
1	0.307	1955	94	24	N.S.	154	21	N.S.
		1957	72	18		176	22	
2	.154	1955	144	—	P=.05	200	—	N.S.
		1957	102	—		167	—	
3	.077	1955	69	29	N.S.	85	40	P<.05
		1957	52	36		56	47	
4	.038	1955	184	92	N.S.	327	167	N.S.
		1957	170	97		284	144	
Combined test routes vs. State totals.		1955	501	3, 788		766	6, 711	P<.02
		1957	396	4, 222	P<.01	683	6, 591	
Combined test routes vs. control sections.		1955	501	145	P<.05	766	228	N.S.
		1957	396	151		683	213	

[1] N.S.=not significant.

Table 4.—Comparison of changes in traffic volumes on test routes Nos. 1 and 4 after increased patrol with traffic volumes on control routes with no increase in patrol

Year	Test route				Control route			
	No. 1		No. 4		No. 1		No. 4	
	Change from expected value	Level of significance [1]	Change from expected value	Level of significance [1]	Change from expected value	Level of significance [1]	Change from expected value	Level of significance [1]
1956	−4.3	N.S.			+6.3	N.S.		
1957	−8.7	P<.07	−2.62	N.S.	+3.2	N.S.	−1.5	N.S.

[1] N.S.=not significant.

These results indicate that for the more discriminating comparison between the matched test and control routes there appeared to be little reliable effect due to enforcement. Regardless of level of patrol, the differences between test and control routes were consistently small. Also, the significant differences found on comparing test routes with the State as a whole had to be interpreted with caution. Although enforcement may have been one of the determining influences, at least two other alternatives are possible. One alternative is the accident frequencies of the test routes which may have been abnormally high prior to the study period. In this case accident frequencies would reasonably show a regression toward the expected value thereafter. This would appear as a significant reduction when compared to the more reliable State figures. A second alternative concerns the large numbers of statewide accidents relative to the test route frequencies. Very large cell frequencies in only one of the two halves of the matrix used to compute the chi-square increases the probability of a significant chi-square, regardless of the magnitude of change in the half of the matrix with the lower cell frequency. Either of these alternatives could account for the anomalous differences among the data shown in the tables.

Traffic volumes and speeds

The results on changes in traffic volume are shown in table 4. For the change in volume over the 7-year period, a trend equation was derived by using the method of least squares. Control limits were derived using the standard error of estimate. The volume in the test year was compared with what was expected on the basis of the trend equation. The level of significance column shown in the table is the probability of the observed change in volume being due to mere chance. As may be seen, on neither test nor control routes did the volume change significantly. Consequently, it is reasonable to conclude that enforcement did not cause a significant diversion of traffic regardless of the intensity of enforcement.

A sample of approximately 3,000 vehicle speeds was obtained for both 1955, the last prepatrol year, and 1957, the first full year that patrol units were operating on all test sections. A control sample of approximately the same size was obtained for the same 2 years. The control sample was obtained from highways having characteristics of design and volume similar to the test sections, but far enough removed from the test routes to minimize any carryover effect from enforcement. The speed samples were taken by the highway department as part of their annual survey of vehicle speeds and were obtained during daylight hours and while traffic was moving freely. All samples were taken at points where speed limits were 65 m.p.h. for passenger cars and 45 m.p.h. for trucks.

The statistics of the speed distributions for passenger cars and trucks are shown in table 5. In general, the mean speed for both trucks and passenger cars decreased for both test and control routes from 1955 to 1957. In view of this, it is difficult to place much reliance in a hypothesis that increased enforcement reduced the average speed of vehicles. While there were definite decreases in average speeds in 1957, as compared with 1955, such decreases followed a similar pattern throughout the State and could not be related to an increase in enforcement activity.

The percentage of drivers exceeding the speed limit followed the same pattern as the mean speeds. There were decreases from 1955 to 1957 on both experimental and control routes. However, the changes on the test routes had no clear-cut relation to enforcement level. Thus, although there were decreases in the proportion of vehicles over the speed limit, there was little evidence to support the hypothesis that this was due to the increased patrol.

Difference in variance

The variances on the test and control routes were compared using an F test. The F test is a test of the differences between variances (S^2) and is the ratio of the larger variance to the smaller. The data are shown for passenger cars and trucks in table 6. It should be noted that for passenger cars there was a significant reduction in the variability in speed in three of the four test routes but in none of the control sections. For trucks only, test routes Nos. 1 and 4 show a significant decrease. However, the control route for No. 4 also showed a significant change from 1955 to 1957. It should be further noted that the magnitude of the F test ratios decreased for the first three test routes as the amount of enforcement decreased. This occurred for both passenger cars and trucks. A plot of the variance ratio against enforcement level is shown in figure 2. For the enforcement levels of zero shown in the curve, the values of the ratio of speed variances were obtained by averaging the ratios for the control sections as shown in table 6. This may indicate that one effect of enforcement is to cause drivers to cluster more nearly about the mean speed and to do so in some relation to the frequency with which drivers as a whole encounter police patrol.

Discussion of the Data

The results of this study as analyzed here indicate quite clearly that any effects of enforcement on traffic behavior are quite indirect. In part, this was due to the measures used to infer the effects of police patrol. Accidents are an obviously inefficient measure. They are rare events in both time and space, having complex causes. The variety of contingent factors underlying their occurrence limits their use as a reliable criterion of improvement techniques except under the most restricted of conditions. In the normal field situation this is rarely the case. In addition, the number of interacting factors involved in the generation of an accident are generally so large that the effect of any one variable will be negligible. Such complexity almost insures defeat in the isolation of the influence of enforcement on traffic behavior.

Table 5.—Speeds of passenger cars and trucks on test and control routes for 1955 and 1957

Route No.	Test routes						Control routes					
	1955			1957			1955			1957		
	Mean	Standard deviation	Sample size	Mean	Standard deviation	Sample size	Mean	Standard deviation	Sample size	Mean	Standard deviation	Sample size
Passenger cars:	*M.p.h.*	*M.p.h.*		*M.p.h.*	*M.p.h.*		*M.p.h.*	*M.p.h.*		*M.p.h.*	*M.p.h.*	
1	60.64	7.88	809	56.79	4.74	911	55.90	7.86	410	51.80	8.76	419
2	54.56	8.78	1,512	54.58	7.54	454	50.23	8.61	898	48.15	8.35	1,127
3	52.53	7.08	573	46.68	7.34	879	57.04	8.47	780	54.62	8.89	661
4	57.52	7.25	910	51.74	6.60	916	49.32	7.81	505	46.35	7.94	1,189
Trucks:												
1	48.21	4.31	140	42.50	2.72	135	49.17	4.65	93	41.75	3.97	127
2	46.51	6.16	268	43.45	5.52	120	48.75	7.37	160	42.62	6.49	125
3	51.25	5.08	180	44.07	5.54	147	53.72	6.62	192	44.37	6.01	147
4	47.58	5.84	197	45.40	4.82	186	49.12	5.12	77	43.33	6.86	186

Enforcement influence on speed

It would seem more reasonable to examine a feature of traffic behavior more directly sensitive to events occurring in traffic and which are under the conscious control of the driver. This is generally the case with vehicle speed, for this is one of the basic modes of vehicle control available to the driver and should be, therefore, one to which he is most responsive. It would seem reasonable that speed would be a primary dimension that a driver would employ to compensate for any pressure he felt from the presence of enforcement. Thus, speed would be one basic variable for detecting the influences of enforcement on traffic behavior.

The mechanisms by which police patrol may operate to influence speed changes are obviously complex. However, such social symbols, i.e. police, may be expected to function at least as agents for inducing conformity in social behavior. In terms of drivers' speed responses this may take one of two forms. First, those drivers who are exceeding the speed limit would be the ones most responsive to patrol. Therefore, it is reasonable to predict that the proportion of the drivers exceeding the speed limit should decrease. Comparison of the test and control routes for the study years, 1955 and 1957, indicate that such a hypothesis is not supported. Although there was a reduction in the proportion of drivers exceeding the speed limits, this reduction was similar on both control and test routes.

The second hypothesis is that patrol influences all drivers. In the case of speed, it is most reasonable to believe that any movement toward conformity would take the form of a general regression of all speeds toward the mean of the traffic stream. This would naturally lead to a reduction in the variance of the speed distribution. The data obtained in this study indicate that this second hypothesis is quite reasonable. From table 6 it may be seen that for passenger cars in three of the four test routes, the amount of reduction in variability of speed seems to decrease in direct proportion to the amount of enforcement.

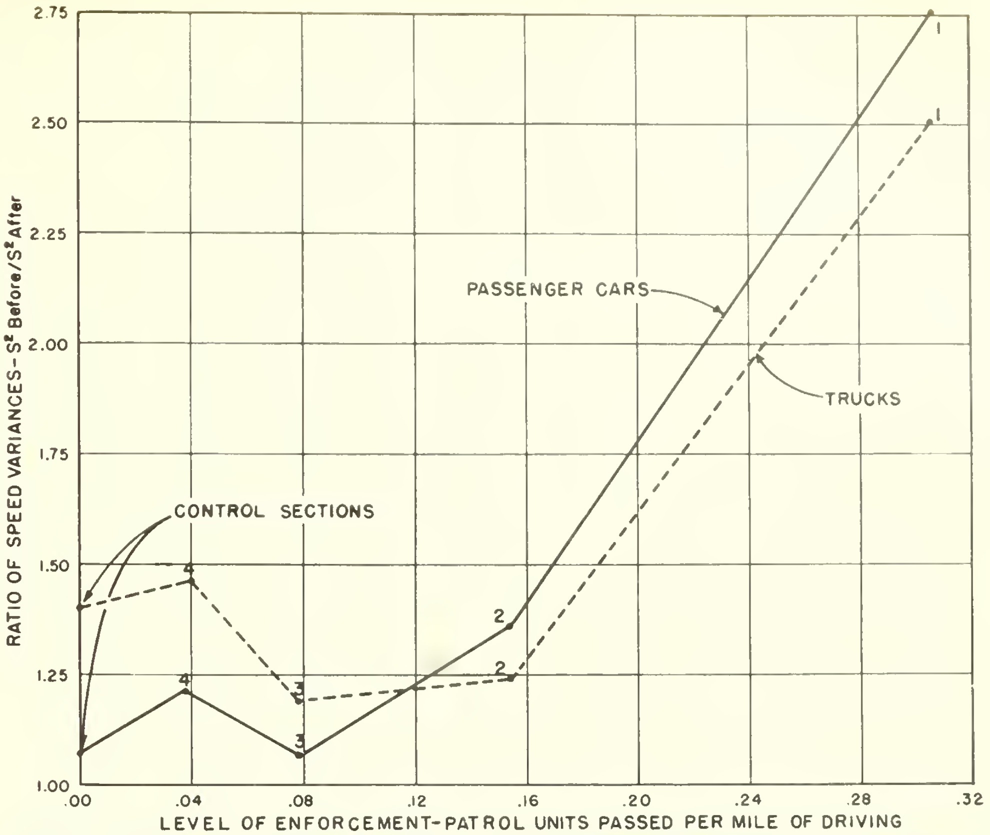

Figure 2.—The change in the variance of speeds under different levels of enforcement (Numbers 1–4 represent the test routes).

These data are of interest in light of some recent findings on the relation between accident rate and travel speed (*4*). It was shown that the accident involvement rate rose rapidly for vehicle speeds less than 40 m.p.h. These speeds were observed on roads on which the average speed was 52 m.p.h. A similar, but less conspicuous, increase was observed for speeds in excess of 75 m.p.h. on the same roads. Thus, drivers operating in the extremes of the speed distribution tended to come into traffic conflict more frequently (on a rate basis) than those travelling at or near the mean. This suggests that when the variance of the speed distribution is high, the frequency and magnitude of the speed difference judgments that drivers must make are also high. Any corrective action these judgments require may be similarly extreme. Consequently, errors of judgment or response may be magnified sufficiently to generate an accident. In light of this analysis, the present data are suggestive of a way in which enforcement may influence accidents. For if, in fact, police patrol causes a reduction in the variance of speed, then it would reduce the frequency of extreme judgments and decisions in driving. Thus, the driver would be forced to make fewer extreme responses with their attendant possibilities for extreme errors, i.e., accidents.

If this is, in fact, one way in which police patrol does function, then it is understandable why little reliable evidence exists relating accidents and level of enforcement activity. First of all, policing would be related to accidents only through second order or mediating processes, e.g., speed. Its effects would then be highly indirect. Secondly, speed is not a unique determinant of accidents, but only one of many. Thus, the proportion of all accidents in which speed differences are an important determinant may be a relatively small part of the total accident distribution. Consequently, the accident reduction occurring when enforcement is increased will vary in each situation depending upon the type of interactions among the variables determining the accident distribution. From the present data, it would appear that speed variability, on the test routes used, represented only a small factor in the total accident process.

It should be pointed out that the results of this study do not offer clear proof of the above analysis. The data are at best suggestive. Several considerations prevent an unequivocal interpretation of the data. One consideration is the obvious discrepancy in the data for test route No. 4 which had the lowest level of enforcement. From table 6 it may be seen that this route showed a significant reduction in speed variability. From the available data it is difficult to explain this change. However, test route No. 4 was the longest of the four

(*Continued on page 124*)

Table 6.—Differences between variances (S^2) of passenger cars and trucks for 1955 and 1957

Route No.	Test routes				Control routes			
	S^2(1955)	S^2(1957)	F	Probability [1]	S^2(1955)	S^2(1957)	F	Probability [1]
Passenger cars:								
1	62.09	22.46	2.76	<.01	61.77	76.73	1.18	N.S.
2	77.08	58.37	1.35	<.01	74.13	69.72	1.06	N.S.
3	50.12	53.87	1.07	N.S.	71.74	79.03	1.02	N.S.
4	52.56	43.56	1.21	<.01	60.99	63.04	1.03	N.S.
Trucks:								
1	18.57	7.39	2.51	<.01	21.62	15.76	1.38	N.S.
2	37.94	30.47	1.24	N.S.	54.31	42.12	1.29	N.S.
3	25.80	30.69	1.19	N.S.	43.82	36.12	1.21	N.S.
4	34.10	23.23	1.47	<.01	26.21	47.06	1.78	<.01

[1] N.S.=not significant.

Appraisal of O-D Survey Sample Size

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported[1] by ARTHUR B. SOSSLAU and GLENN E. BROKKE,
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The objective of this research was to determine the relationship between home-interview type of origin and destination survey sampling rates and the error that can be expected in the volumes accumulated from such a survey.

The data analyzed were taken from the home-interview phase of the 1957 Phoenix-Maricopa County, Ariz., traffic study. Since individual zone-to-zone movements have insufficient volume to be reliable indicators, a method called "trip tracing" was used to accumulate individual zone-to-zone trip volumes for statistical study.

The results indicate that the accuracy of accumulated trip volumes is considerably less than the accuracy predicted by a purely theoretical approach. However, the results agree qualitatively with theory in that the accuracy of trip volumes varies with the square root of sample size and very nearly with the square root of the volume. The equations developed should allow the selection of a sample rate commensurate with the funds available and the degree of accuracy required.

TRAVEL BY INDIVIDUALS has the characteristic of being habitual. Furthermore, the travel habits of different individuals are similar for the many types of trips generally made. For these reasons, sampling procedures, such as the home-interview type of origin and destination survey developed by the Bureau of Public Roads, can be used to determine travel information for a metropolitan area.

The accuracy of information developed from a home-interview survey, and indeed from any type of sampling procedure, is to an important degree dependent upon the sampling rate used. Sampling rates employed in origin and destination surveys have usually been based on population and area. In a large, densely populated area a smaller sampling rate is used than in a small, less densely populated area. The sampling rate determined is based, predominantly, on the number of interviews needed to provide a reliable representation of the overall travel in the city. The zone-to-zone movements developed, however, have been used to present a picture of travel desires and to aid in the location of needed transportation facilities.

The results of an origin-destination survey are also accumulated on, or assigned to, proposed highway facilities for design considerations. How accurate are these accumulated values that have been expanded from the survey sample to represent average daily movements? The purpose of the study, herein reported, was to determine the relationship between the home-interview type of origin and destination survey sampling rates and the accuracy of volumes that are accumulated therefrom.

Measures of the accuracy of accumulated volumes have been determined for various sampling rates and predictive equations developed for determining the sampling rate needed for a desired accuracy at any volume. Since sample rate affects the accuracy of volumes used for design purposes, an estimate can be made of the frequency that facilities so designed will be loaded beyond capacity or conversely, the frequency that such facilities will have more lanes than are actually required.

Source of Data

The data used for this study were developed from the home-interview phase of the 1957 Phoenix-Maricopa County, Ariz., traffic survey. The survey area covered 225 square miles and contained a population of 397,000 persons. Of the survey area, the city of Phoenix occupied 36.3 square miles and included 192,500 persons. The survey area was divided into 135 zones, which were smaller in area in the downtown section of Phoenix and larger in the outlying areas.

The sample rate used in the home-interview phase of the origin and destination survey was 1 in every 15 dwelling units, or 6.67 percent. This rate resulted in a total sample of 8,743 dwelling units. The trips analyzed were those having both origin and destination within the survey area. All trips, regardless of mode of travel, were considered. The total expanded number of internal trips was 839,398. Separated by mode, it was as follows:

Automobile drivers	548, 439
Automobile, truck, and taxi passengers	233, 536
Bus passengers	57, 423

The survey personnel had checked the expanded household data against the 1950 decennial census and the 1953 special census with satisfactory results. Also the expanded travel data had been compared with two screenline ground counts made during the survey. The survey data accounted for 89.3 percent of the vehicles crossing the screenlines from 6 a.m. to 10 p.m. on an average weekday.

Method of Study

Individual zone-to-zone movements obtained from a home-interview type of origin and destination survey cannot be used as the basis for sound conclusions regarding sample size due to the predominance of small volume movements between many zones.[2] Any sample size, within the limits of economic feasibility, would be too small to produce accurate expanded values for these small movements; but the individual zone-to-zone movements do not necessarily have to be accurate if their summation reasonably represents travel accumulations throughout a city.

A method referred to as trip-tracing,[2] has provided the means for determining and checking distribution of travel throughout an area. By this method zone-to-zone movements are traced across a city in a straight line between origin and destination. After the equation of the straight line between the two zones has been obtained, the points of intersection of this line with previously established gridlines are determined. The volume of trips being traced is added to volumes previously accumulated, for other zone-to-zone movements, at the section of the gridline being crossed. After all zone-to-zone movements have been similarly traced across the city, the result is the accumulated number of trips per section of gridline.

The length of the sections of gridline, in which the volumes are accumulated, is chosen in accordance with the size of the accumulations desired. If large accumulated volumes are desired, large sections are used, and vice versa.

A grid system was superimposed over the Phoenix-Maricopa County traffic study area and is illustrated in figure 1. The southwest corner of the city was designated as origin of the rectangular coordinate system. North-south gridlines were placed at 2, 3, 4, 5, 5.5, 6, 7, 8, 9, 10, 12, and 14 miles from the origin, and east-west gridlines at 4, 6, 7, 8, 8.3, 9, 10, 11, 12, 13, and 15 miles from the origin. The closer spacing of gridlines was at the more densely populated part of the study area. Each gridline was broken into ¼-mile sections for volume accumulation purposes, as shown diagrammatically in the upper right corner of figure 1. The north-south gridlines were 17 miles long and the east-west lines, 16 miles long. This resulted in 1,520 ¼-mile sections (17 miles × 4 sections per mile × 12 gridlines + 16 miles × 4 sections per mile × 11 gridlines). The volumes accumulated on the ¼-mile sections ranged from 0 to 35,000 trips.

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[1] This article was presented at the 39th Annual Meeting of the Highway Research Board, Washington, D.C., January 1960.

[2] *Evaluating trip forecasting methods with an electronic computer*, by Glenn E. Brokke and William L. Mertz. PUBLIC ROADS, vol. 30, No. 4, Oct. 1958, pages 77–87.

Features of the procedures that have been described are as follows:

1. The result of the trip-trace accumulation is a spatial distribution of trips throughout the city representing the travel desires of the population. The trips are traced in such a manner that each zone-to-zone movement is made by the most desirable path—a straight line.
2. Long trips are weighted more heavily than short trips since more gridlines are crossed. This is analogous to long trips using more of a road network than shorter trips.
3. The resulting accumulation of trips presents a picture of travel analogous to accumulations of trips on a street system.

The trip-tracing procedure, much too lengthy for manual computation methods, is readily adaptable to computation on an electronic computer of medium size. Therefore, a computer program was developed to trace the Phoenix-Maricopa County trips across the survey area. Computer programs were also developed to handle the various statistical computations that were necessary for this study. A simplified flow chart of the trip-tracing program is shown in figure 2. Generally, the following computations were carried out:

Table 1.—Volume group summary

Volume group No.	Volume group range (average daily person trips)	Number of sections having volumes within range [1]	Mean volume of group [2] (average daily person trips)
1	0.1– 199.9	268	91.1
2	200.0– 399.9	88	283.2
3	400.0– 599.9	69	486.7
4	600.0– 799.9	45	682.0
5	800.0– 999.9	50	893.3
6	1,000.0– 1,499.9	80	1,244.5
7	1,500.0– 1,999.9	56	1,763.4
8	2,000.0– 2,499.9	58	2,257.4
9	2,500.0– 3,499.9	86	2,998.6
10	3,500.0– 4,499.9	62	4,001.4
11	4,500.0– 5,499.9	42	5,020.9
12	5,500.0– 7,999.9	67	6,759.3
13	8,000.0– 9,999.9	41	8,961.8
14	10,000.0–14,999.9	31	12,401.3
15	15,000 and over	28	21,689.9

[1] The number of sections do not equal 1,520 since some sections did not have any trips traced across them.

[2] The mean volume was determined from the total sample.

1. From the coordinates of the two zones being handled, the equation of the straight line passing between them was determined.
2. The points of intersection of the line and the north-south and east-west gridlines, to the nearest quarter of a mile, were determined.
3. The number of trips being traced were then added to the volumes previously accumulated in the computer memory locations representing the 1,520 ¼-mile sections.

All of the internal Phoenix-Maricopa County home-interview trip cards, representing the expanded results of the 1-in-15 dwelling-unit sample, were processed through the trip-tracing program to determine the spatial distribution of trips throughout the survey area. The original deck of cards was then sorted by sample number into two one-half subsamples, each representing a 1-in-30 dwelling-unit sample. Each one-half subsample was then separately run through the computer to determine the spatial distribution of the two 1-in-30 dwelling-unit samples. Similarly, the original deck was systematically stratified by sample number into 3 one-third subsamples, and 10 one-tenth subsamples, and each subsample processed through the trip-tracing program.

Statistical procedure

Use of the total Phoenix-Maricopa County traffic survey data resulted in accumulated average daily volumes, in the 1,520 ¼-mile sections, ranging from 0 to 35,000 person-trips. Similarly, each of the subsamples processed through the trip-tracing program resulted in accumulated volumes in each section. It should be noted that, as each subsample zone-to-zone movement was traced, the volume was expanded to represent actual movement. For example, as each one-third subsample trip card was processed, the trip factor on the card was multiplied by three.

The data resulting from the trip-tracing program were analyzed by comparing, on a ¼-mile section basis, the expanded subsample accumulations against the total sample accumulations. It was not, however, considered practical to report the error for each section. Instead, the 1,520 ¼-mile sections

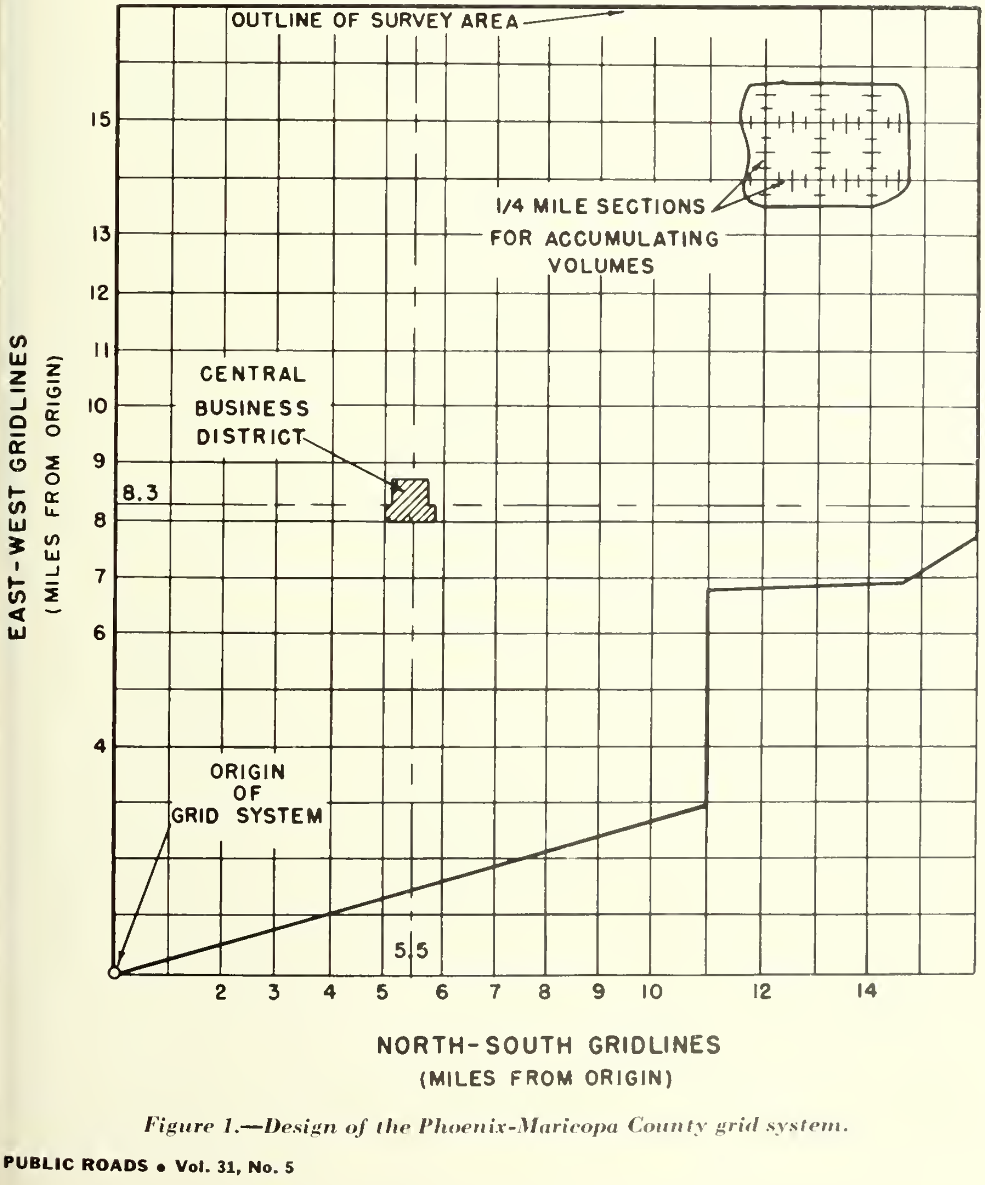
OUTLINE OF SURVEY AREA
1/4 MILE SECTIONS FOR ACCUMULATING VOLUMES
CENTRAL BUSINESS DISTRICT
ORIGIN OF GRID SYSTEM
8.3
5.5
15
13
12
11
10
9
8
7
6
4
2
3
4
5
6
7
8
9
10
12
14
EAST-WEST GRIDLINES (MILES FROM ORIGIN)
NORTH-SOUTH GRIDLINES (MILES FROM ORIGIN)

Figure 1.—Design of the Phoenix-Maricopa County grid system.

were stratified into 15 volume groups, and the individual errors were accumulated and summarized for each volume group. Such a process produced, for each of the 3 subsamples tested, 15 errors and the average volume at which the error occurred. The range, the number of sections, and the average volume of each volume group are shown in table 1. The sections of gridline were stratified into volume groups in accordance with the volumes accumulated in the sections from the expanded total sample.

The summarization of results, per volume group, for each subsample consisted of determining the differences in volumes accumulated per section from the original sample and the subsample, squaring the differences and accumulating the results of the squaring. The resulting summation was then divided by the number of sections in the volume group and the square root of the quotient taken. The result of this procedure is the root-mean-square error (RMS error) of the subsample as compared with the total sample. The equation for determining this error is:

$$\text{RMS error} = \sqrt{\frac{\sum_{i=1}^{i=n} (V_{ss} - V_s)^2}{n}}$$

Where

V_{ss} = volume accumulated in section i from subsample.
V_s = volume accumulated in section i from total sample.
n = number of sections in volume group.

The RMS error is comparable, statistically, to the standard deviation of a group of values around their mean. For example, if the RMS error for a one-third subsample volume compared with the original sample volume is 50 person-trips, one would make little error by assuming that two-thirds of the expanded volumes obtained from the subsample would lie within 50 person-trips of the total sample volume.

RMS errors were developed for each subsample. That is, a RMS error was developed for each of the one-half subsamples, for each of the one-third subsamples, and for each of the one-tenth subsamples. However, since little could be gained by reporting and analyzing each subsample RMS error, a mean error was determined from the 3 one-third subsamples, from the 2 one-half subsamples, and from the 10 one-tenth subsamples.

A RMS error was computed for each volume class, and the percent root-mean-square error was determined by dividing the numerical error by the average volume of the volume class being considered. The results of the one-third subsample comparison are presented in table 2.

Results of the Study

The results of the trip-tracing program and the statistical procedure used are the percent root-mean-square (percent RMS) errors for the one-half, the one-third, and the one-tenth subsamples, each measured against the total Phoenix-Maricopa County sample. These results are shown in table 3 for each volume group.

As was to be expected, the percent RMS error for any particular volume group is invariably greatest for the smallest subsample rate, and vice versa. For example, for an average volume of 1,763 trips, volume group No. 7, the percent RMS error for the one-half subsample was 15.3; for the one-third subsample, 23.5; and for one-tenth subsample, 49.4. In addition, the percent RMS error, for each subsample, decreased as the volume increased.

It should be understood that the one-half subsample errors are in reality the percent RMS errors between a 1-in-30 and a 1-in-15 dwelling-unit sample. Likewise, the one-third subsample rate is actually a 1-in-45 dwelling-unit sample and the one-tenth subsample is a 1-in-150 dwelling-unit sample, the error in each case being measured against the 1-in-15 sample.

The values tabulated in table 3 can be used in the following manner. If it is desired to estimate trips from a home-interview survey at the 4,000 average daily person-trip level, volume group No. 10, for some specific design

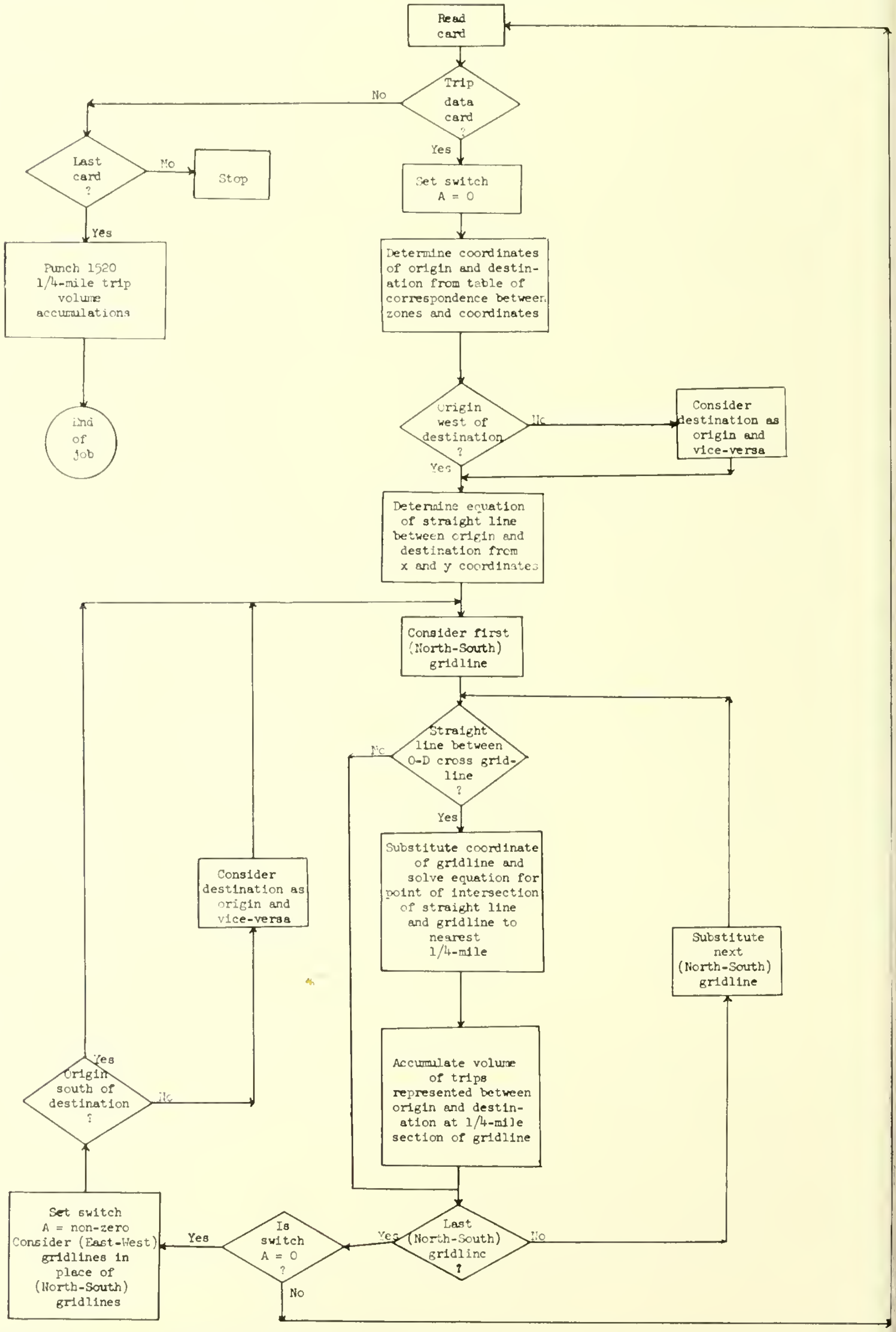
Read card
Trip data card ?
No
Yes
Last card ?
No
Stop
Yes
Punch 1520 1/4-mile trip volume accumulations
End of job
Set switch A = 0
Determine coordinates of origin and destination from table of correspondence between zones and coordinates
Origin west of destination ?
No
Consider destination as origin and vice-versa
Yes
Determine equation of straight line between origin and destination from x and y coordinates
Consider first (North-South) gridline
Straight line between O-D cross gridline ?
No
Yes
Substitute coordinate of gridline and solve equation for point of intersection of straight line and gridline to nearest 1/4-mile
Substitute next (North-South) gridline
Consider destination as origin and vice-versa
Accumulate volume of trips represented between origin and destination at 1/4-mile section of gridline
Yes
Origin south of destination ?
No
Set switch A = non-zero Consider (East-West) gridlines in place of (North-South) gridlines
Yes
Is switch A = 0 ?
No
Yes
Last (North-South) gridline ?
No

Figure 2.—Generalized electronic computer flow chart for tracing trips across a grid system.

Table 2.—Results of one-third subsample comparison with total sample

Volume group No.	Mean volume of group (total sample)	Mean volume [1]			Mean error [2]			RMS error [3]			Percent RMS error [4]			Average RMS error of three samples	
		(1st) One-third subsample	(2d) One-third subsample	(3d) One-third subsample	(1st) One-third subsample	(2d) One-third subsample	(3d) One-third subsample	(1st) One-third subsample	(2d) One-third subsample	(3d) One-third subsample	(1st) One-third subsample	(2d) One-third subsample	(3d) One-third subsample	Absolute error: $\frac{\text{RMS error (1st)+(2d)+(3d)}}{3}$	Percent error: $\frac{\text{Absolute error}}{\text{Original Sample Volume}}$
1	91.1	83.2	90.6	99.4	−7.9	−0.4	+8.3	77.5	75.1	78.9	85.07	82.44	86.61	77.2	84.74
2	283.2	231.2	309.5	308.9	−52.0	+26.3	+25.7	160.5	153.5	168.3	56.67	54.20	59.43	160.8	56.78
3	486.7	392.9	545.0	522.2	−93.8	+58.3	+35.5	205.6	237.4	225.7	42.24	48.78	46.37	222.9	45.80
4	682.0	623.2	710.9	711.9	−58.8	+28.9	+29.9	307.8	253.6	295.6	45.13	37.18	43.34	285.7	41.89
5	893.3	749.9	977.1	952.7	−143.3	+83.8	+59.5	300.5	306.4	267.8	33.64	34.30	29.98	291.6	32.64
6	1,244.5	1,153.8	1,325.3	1,254.2	−90.6	+80.9	+9.7	308.9	335.2	349.7	24.82	26.93	28.10	331.3	26.62
7	1,763.4	1,772.5	1,883.2	1,634.4	+9.1	+119.8	−128.9	390.8	427.0	424.9	22.16	24.21	24.10	414.2	23.49
8	2,257.4	2,123.7	2,631.0	2,287.5	−133.7	+373.6	+30.1	470.6	537.9	391.4	20.85	23.83	17.34	466.6	20.67
9	2,998.6	2,886.6	3,118.3	2,990.9	−112.0	+119.7	−7.7	513.6	578.4	503.6	17.13	19.29	16.79	531.9	17.74
10	4,001.4	3,938.8	3,995.6	4,069.8	−62.6	−5.8	+68.4	666.1	654.7	644.7	16.65	16.36	16.11	655.2	16.37
11	5,020.9	4,903.9	5,136.4	5,022.5	−117.0	+115.5	+1.6	727.0	667.7	729.5	14.48	13.30	14.53	708.1	14.10
12	6,759.3	6,490.2	6,628.3	7,159.4	−269.1	−131.0	+400.1	774.5	751.7	922.5	11.46	11.12	13.65	816.2	12.08
13	8,961.8	8,896.1	8,923.1	9,066.4	−65.7	−38.8	+104.5	878.5	780.4	910.2	9.80	8.71	10.16	856.4	9.56
14	12,401.3	12,285.2	12,461.4	12,457.2	−116.1	+60.1	+55.9	909.7	913.7	1,156.3	7.34	7.37	9.32	993.2	8.01
15	21,689.9	21,555.2	22,646.2	21,468.2	−134.7	+356.3	−221.7	1,741.0	1,808.9	1,655.6	8.03	8.34	7.63	1,735.2	8.00

[1] Mean volume $= \dfrac{\Sigma\binom{\text{Segment volumes}}{\text{in volume group}}}{\text{Number of sections}}$

[2] Mean error $= \dfrac{\Sigma\left[\binom{\text{Subsample}}{\text{volume}}-\binom{\text{Original sample}}{\text{volume}}\right]}{\text{Number of sections}}$

[3] RMS error $= \sqrt{\dfrac{\Sigma\left[\binom{\text{Subsample}}{\text{volume}}-\binom{\text{Original sample}}{\text{volume}}\right]^2}{\text{Number of sections}}}$

[4] Percent RMS error $= \dfrac{\text{RMS error}}{\text{Mean volume}}$

purpose, the use of a 1-in-30 dwelling-unit sample would produce a volume that is within 11.7 percent of the value that would have been obtained with a 1-in-15 sample two-thirds of the time. Likewise, the use of a 1-in-45 dwelling-unit sample would result in a volume that is within 16.4 percent of the value obtained by a 1-in-15 sample two-thirds of the time. If the probability of being within the 1-in-15 sampling rate volume 95 percent of the time is desired, 2 times the percent RMS error would be used. An expectancy of 99 percent would require 3 times the percent RMS error.

By using the values tabulated in table 3, the expected results of a 1-in-30, a 1-in-45, and a 1-in-150 dwelling-unit sample can be compared with that of a 1-in-15 sample. However, the primary purpose of this study was to determine the error between the volume determined from any dwelling-unit sample and the actual volume; the actual volume being the average daily person-trips measured over the study period. It is evident that only through an overwhelming expenditure of time and money could every person in a city be interviewed every day during the study period. However, through statistical procedures, using the comparisons of the 1-in-30, the 1-in-45, and the 1-in-150 dwelling-unit sampling rate with the 1-in-15 sample, an estimate of the error between any size sample and the total population could be determined.

Source of error

The error between a volume determined from any of the subsamples and the true volume consists of two parts: First, the error between the subsample volume and the total Phoenix-Maricopa County sample volume; and second, the error between the total sample volume and the true volume. In statistical computations, for the analysis of variance, the total variance of a group of samples is equal to the "between sample" variance plus the "within sample" variance:

$$\sigma^2_{\text{total}}=\sigma^2_{\text{within}}+\sigma^2_{\text{between}} \qquad (1)$$

where: σ^2=variance.

The percent RMS errors computed for this study, as mentioned previously, are statistically comparable to the standard deviation of a group of values about their mean. Therefore, an equation for relating percent RMS errors, comparable to equation (1) above is:

$$E^2_{ss\text{-}o}=E^2_{ss\text{-}s}+E^2_{s\text{-}o} \qquad (2)$$

Where:

$E_{ss\text{-}o}$=percent RMS error of subsample volume measured against true volume.

$E_{ss\text{-}s}$=percent RMS error of subsample volume measured against total sample volume.

$E_{s\text{-}o}$= percent RMS error of total sample volume measured against true volume.

An equation for estimating the error for a sample from the error found for another independently selected sample is:

$$E_{ss\text{-}o}=E_{s\text{-}o}\sqrt{\frac{N_s}{N_{ss}}} \qquad (3)$$

Where:

N_s=number of interviews taken in original survey.

N_{ss}=number of interviews represented in subsample.

Assuming that equation (3) is an acceptable approximation to the situation being considered, and by combining equations (2) and (3) the following equation results:

$$\frac{N_s}{N_{ss}}\times E^2_{s\text{-}o}=E^2_{ss\text{-}s}+E^2_{s\text{-}o}$$

or

$$E^2_{s\text{-}o}=\frac{E^2_{ss\text{-}s}}{\dfrac{N_s-N_{ss}}{N_{ss}}}$$

therefore:

$$E_{s\text{-}o}=\frac{E_{ss\text{-}s}}{\sqrt{\dfrac{N_s}{N_{ss}}-1}} \qquad (4)$$

Considering equation (4) for determining the percent RMS error between the total sample volume and the actual volume, from the error determined from the one-tenth subsample, equation (4) then becomes:

$$E_{s\text{-}o}=\frac{E_{1/10\text{-}s}}{\sqrt{\dfrac{N_s}{1/10N_s}-1}}=\frac{E_{1/10\text{-}s}}{\sqrt{10-1}}$$

$$E_{s\text{-}o}=\frac{E_{1/10\text{-}s}}{3} \qquad (5)$$

Likewise, the equations for determining the error of the total sample from the one-half and one-third subsample errors are:

$$E_{s\text{-}o}=\frac{E_{1/2\text{-}s}}{\sqrt{1}}=E_{1/2\text{-}s} \qquad (6)$$

$$E_{s\text{-}o}=\frac{E_{1/3\text{-}s}}{\sqrt{2}}=\frac{E_{1/3\text{-}s}}{1.41} \qquad (7)$$

Stability of error computation

Equations (5), (6), and (7) were used to determine independent estimates of the percent RMS errors in the total sample volumes,

Table 3.—Percent root-mean-square error of subsample volume as measured against total sample volume, by different sample sizes

Volume group No.	Percent RMS error of subsample volume measured against total sample mean volume of group		
	One-balf original sample	One-third original sample	One-tenth original sample
1	67.5	84.7	193.0
2	41.4	56.8	120.7
3	28.4	45.8	88.2
4	27.6	41.9	80.2
5	23.1	32.6	67.7
6	18.0	26.6	55.7
7	15.3	23.5	49.4
8	13.4	20.7	45.0
9	14.2	17.7	36.6
10	11.7	16.4	33.8
11	10.2	14.1	29.0
12	9.3	12.1	26.4
13	6.7	9.6	21.4
14	5.4	8.0	18.5
15	3.8	8.0	16.0

Table 4.—Estimated percent root-mean-square error of original sample volume as measured against population volume

Volume group No.	Estimated percent RMS error from—			
	One-half original sample	One-third original sample	One-tenth original sample	Mean
1	67.5	59.9	64.3	63.9
2	41.4	47.3	40.2	43.0
3	28.4	32.4	29.4	30.1
4	27.6	29.7	26.7	28.0
5	23.1	23.1	22.6	22.9
6	18.0	18.8	18.6	18.5
7	15.3	15.9	16.5	15.9
8	13.4	14.7	15.0	14.4
9	14.2	12.6	12.2	13.0
10	11.7	11.6	11.3	11.5
11	10.2	10.0	9.7	10.0
12	9.3	8.6	8.8	8.9
13	6.7	6.8	7.1	6.9
14	5.4	5.7	6.2	5.8
15	3.8	5.7	5.4	5.0

measured from the actual volume, from the one-tenth, one-half, and one-third subsample errors, respectively. If each of these independent calculations produced consistent estimates of the RMS error for the 1-in-15 sample, it appears reasonable to assume that other sampling rates would be equally consistent. These independent estimates of the error in the total sample are shown in table 4.

A comparison of the estimated percent RMS errors of the 1-in-15 dwelling-unit sample, for each volume group, shows little variation. The mean of the three estimates was, therefore, determined and plotted on logarithmic paper (fig. 3). A least-squares fit determined from the points is also shown in figure 3. The equation of the developed line is:

$$\text{Percent RMS error of 1-in-15 dwelling-unit sample}=\frac{629.0}{\text{volume}^{.4884}} \qquad (8)$$

The coefficient of correlation developed is approximately 1, indicating an almost perfect functional relationship between the two variables considered. In other words, the variation in percent RMS error is explained almost entirely by the variation in volume. Since this is a logarithmic relationship the standard error of estimate for the line is a constant percent of the estimated value rather than a constant percent error. The error, which is 5.65 percent of the estimated values, means that at a volume of 1,000 trips, the standard error is about 1.2 percent (5.65 percent × 21 percent) and at 10,000 trips the standard error is about 0.4 percent (5.65 percent × 6.8 percent).

Equation (3) can be used, in the following form, for determining the relationship between percent RMS error and volume for any sample rate:

$$E_{i-o}=E_{1/15-o}\times\sqrt{N_i} \qquad (9)$$

Where:

E_{i-o} = error of any sample i.

$E_{1/15-o}$ = error of 1-in-15 dwelling-unit sample.

N_i = number of times the sampling rate of survey i is less than 1-in-15. For example, if a 1-in-60 rate is used, N_i would be 4. If the rate was 1-in-5, N_i would be 1/3.

If a 1-in-1 sample were taken, that is, every person in the city interviewed once, the equation for determining percent RMS error would be:

$$\text{Percent } RMS \text{ error}=\frac{629.0\sqrt{N_i}}{\text{volume}^{.4884}}$$

where: $N_i=1/15$

therefore:

$$\text{percent } RMS \text{ error for 1-in-1 dwelling-unit sample}=\frac{162.4}{\text{volume}^{.4884}} \qquad (10)$$

It should be noted that the error for a 1-in-1 dwelling-unit sample is not zero, since a 1-in-1 sampling rate is not a 100-percent sample for determining average daily traffic during the survey period. Every person in a city would have to be interviewed about his travel for every day during the survey in order to obtain the universe of travel for that period.

Equation (10) can be used for determining the equation for percent RMS error at any sample rate. Simply multiply equation (10) by the square root of the denominator of the sample rate ratio used. For example, if a 1-in-30 home-interview sample rate is used, multiply equation (10) by the square root of 30. Figure 4 presents predictive lines for estimating percent RMS errors for volumes between 100 and 100,000 person-trips per day for various sampling rates.

Comparison of Results

A theoretical approach to the problem of estimating the accuracy of various sample sizes relies on the estimation of the standard deviation in volumes determined from the samples. The theory states that the expected deviation is expressed as follows:

σ = expected deviation
V = Volume
S = percent sample

$$\sigma=\sqrt{V\left(\frac{S}{100}\right)\left(1-\frac{S}{100}\right)}$$

Since the probable volume ($\overline{V}$) is equal t the volume obtained from the survey times th sample rate:

$$\frac{\sigma}{V}=\frac{\sqrt{V\left(\frac{S}{100}\right)\left(1-\frac{S}{100}\right)}}{V\frac{S}{100}}$$

or

$$\text{percent } \sigma=100\sqrt{\frac{100-S}{VS}}$$

A comparison of the errors predicted by th above theory with the relationship develope in this article is shown in table 5 for variou volumes and sample sizes. It can be see that the observed root-mean-square errors a from 1.7 to 1.9 times as great as the err predicted by theory. This difference may b due to nonsampling errors such as response an coding errors. For example, a study made i Cincinnati, Ohio, showed that those inte viewed reported too few trips in some case and too many trips in other cases.

In table 5 it is found that the observed err is almost two times as great as the theoretic error. This means that in order to maintain desired degree of accuracy, it is necessary t increase the sampling rate to almost four tim the rate indicated by the theoretical standa deviation computation.

Use of Results

The curves plotted in figure 4 have been d veloped for total person-trips and can be ut lized to determine the sample rate to be use

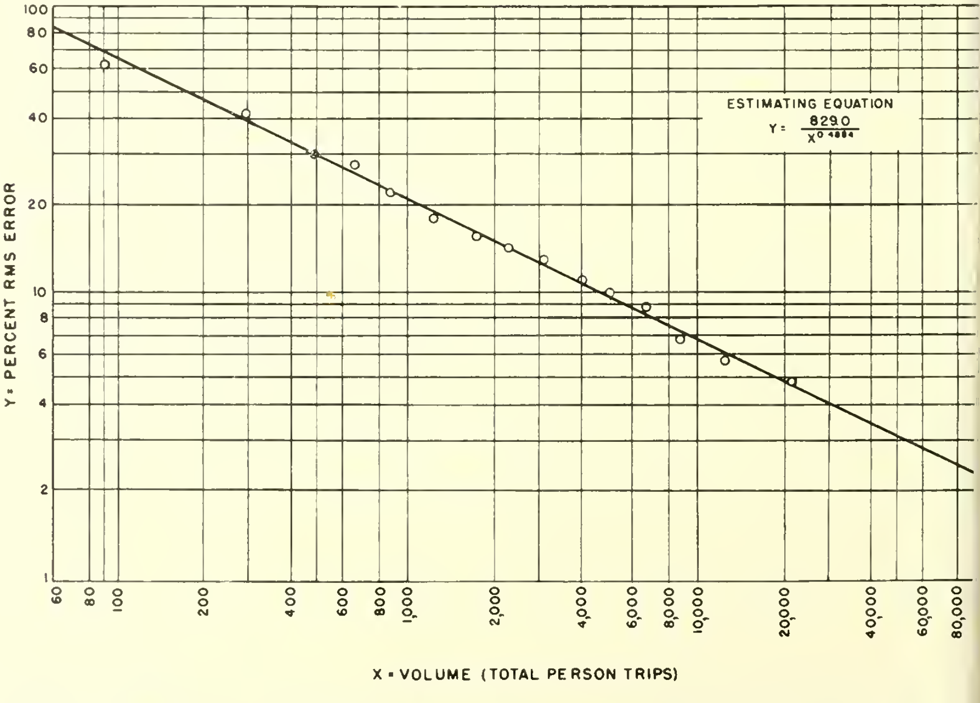

Figure 3.—Relation of percent root-mean-square error and total person trips for 1-in- dwelling-unit sample.

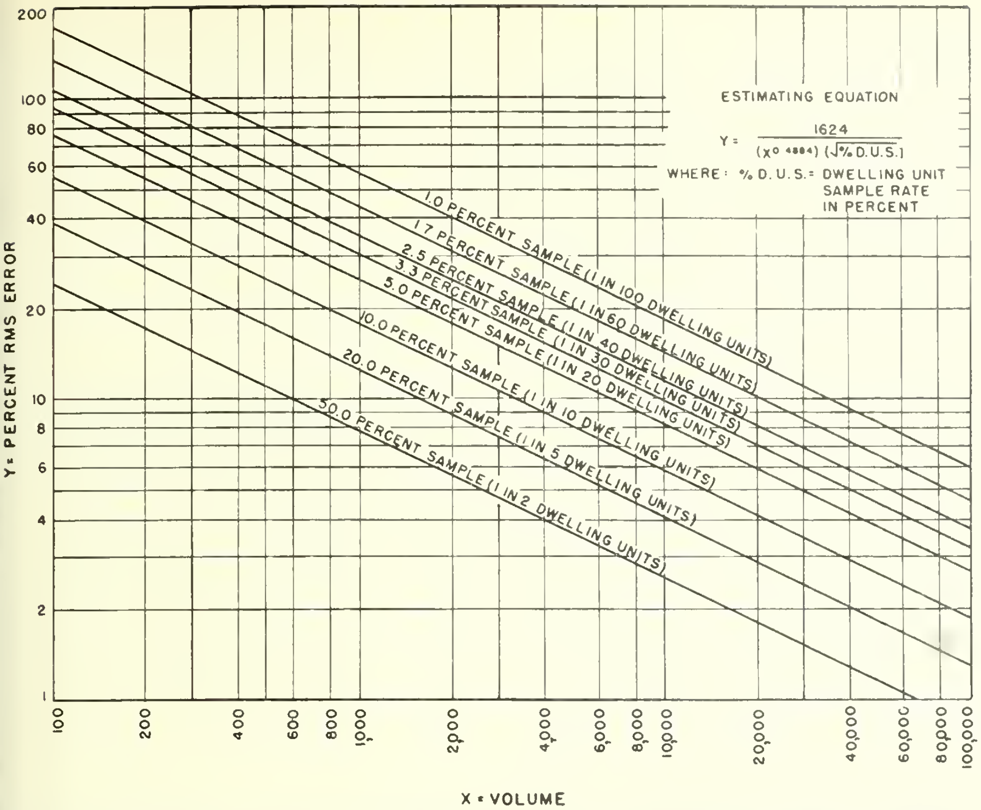

Figure 4.—Relation of percent root-mean-square error and volume for various dwelling-unit sample rates.

when desiring an estimate of volume with a desired degree of accuracy. For example, if it is desired to estimate an average daily volume at the 10,000 person-trip level, and be within 8 percent of the true value 95 percent of the time, from figure 4 at volume equal to 10,000 and percent RMS error equal to 4 percent (8 percent÷2 for 95 percent confidence), it is found that a 20-percent sample should be taken. Similarly, the curves can be used to estimate the error in accumulated volumes after the results of an origin-destination survey are obtained. For rates not plotted, equation (10) would be used in the following form:

$$\text{Dwelling-unit sample rate in percent} = \left[\frac{1,624}{(\text{Percent RMS error})(\text{volume}^{.4884})}\right]^2 \qquad (11)$$

For the problem explained, a dwelling-unit sample rate of 20.4 percent would result from the use of equation (11), indicating that one out of every five dwelling units should be interviewed.

More often than not, the highway engineer is interested in the number of vehicle-trips rather than the number of person-trips—vehicle-trips being the figure used for highway design purposes. Therefore, the question arises: can the curves developed for person-trips be used as an indicator of error for vehicle-trips?

Table 5.—Comparison of observed errors and theoretical sampling errors

Sample rate (percent)	Volume	Observed error [1]	Theoretical sampling error [2]	Observed error : theoretical sampling error
		Percent	*Percent*	
1	100	171.31	99.50	1.7
	1,000	55.64	31.46	1.8
	10,000	18.07	9.95	1.8
	100,000	5.87	3.15	1.9
3	100	98.91	56.87	1.7
	1,000	32.21	17.98	1.8
	10,000	10.43	5.69	1.8
	100,000	3.39	1.80	1.9
4	100	85.65	48.99	1.7
	1,000	27.82	15.49	1.8
	10,000	9.04	4.90	1.8
	100,000	2.94	1.55	1.9
5	100	76.60	43.60	1.8
	1,000	24.90	13.80	1.8
	10,000	8.10	4.36	1.8
	100,000	2.62	1.38	1.9
10	100	51.50	30.00	1.7
	1,000	16.70	9.50	1.8
	10,000	5.43	3.00	1.8
	100,000	1.76	.95	1.9

[1] This is the percent RMS error as developed in this article.
[2] This is the theoretical percent standard deviation error.

The volume of automobile vehicle-trips throughout a city is less than the volume of person-trips, but is similarly distributed. The errors, if developed between the subsamples and total sample for automobile vehicle-trips, should not, therefore, be any different from the errors determined for all person-trips. That is, a percent RMS error for 10,000 automobile vehicle-trips should be no different from the error determined for 10,000 person-trips. The curves presented in figure 4 and the equations developed can therefore be used for total person-trips, automobile vehicle-trips, bus-passenger trips, truck- and taxi-passenger trips, and automobile passenger-trips. The results of this study can also be applied to any home-interview type of origin and destination survey since accumulation of trips and not individual zone-to-zone movements are being compared.

A Traffic Analyzer: Its Development and Application

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported [1] by ASRIEL TARAGIN, Chief,
Traffic Performance Branch,
and RICHARD C. HOPKINS, Chief,
Instrumentation Branch

From the early graphic recording equipment to the modern electronic digital recorders, the Bureau of Public Roads has continuously been seeking faster and more reliable means for recording and analyzing traffic data. This article presents the development of the Bureau's digital speed-placement equipment—the traffic analyzer.

IN THE PAST 20 YEARS significant advances in the development of improved standards for geometric highway design have stemmed primarily from the increasing knowledge of traffic performance, driver behavior, and highway capacity. A better understanding of the many problems has been made possible by the detailed study of the speed and placement of each vehicle in the traffic stream and its relation to the speeds, the lateral placements, and the longitudinal positions of vehicles ahead, behind, and adjacent.

Development of Equipment

In the early stages of research, traffic data were recorded by observers using stopwatches and pavement markings. The necessity for more complete knowledge of the factors determining highway capacity became increasingly evident. Improvement and refinement in the field technique was imperative because more data were needed, with greater precision, as added features of design had to be evaluated, and the results had to be analyzed more quickly.

Increasing use was made of automatic graphic recording equipment as reliable and sophisticated instruments became available for field use. By the late 1930's, the Bureau of Public Roads had developed a reliable speedmeter and placement detector with digital outputs to the time bases available in graphic records.[2] The accuracy and dependability of this equipment was proven in its continuous field use. However, the data had to be scaled from the long rolls of pen-line graphs produced by the equipment before they could be put on punch cards; and the considerable amount of manpower and time required for this greatly delayed the availability of the data for use and limited the volume of data that could be handled. It was only natural, therefore, that means of actual digital recording would be sought, from which machine tabulating cards could be prepared easily, accurately, and quickly, and in large volume.

This thinking became a reality in 1946 when automatic digital recording of data for two lanes of traffic was made available by means of mounting hand-wound solenoids over the keys of a standard adding machine and devising a digital clock to indicate the time of each recording. This assembled equipment proved so successful that a year later it was permanently mounted in a panel truck, and studies were conducted with the equipment in many States throughout the country. With minor improvements in components, but no changes in the basic design, this 2-lane digital speed-placement equipment served well the Bureau of Public Roads, the cooperating State highway departments, and other agencies for 10 years.

In 1956 the recording capabilities of the digital speed-placement equipment were increased to cover four lanes of traffic. The time was then opportune to improve some points of basic design which the availability of recently developed electronic components made feasible. The traffic analyzer was developed by Bureau engineers, modernizing and redesigning the older speed-placement recording equipment.

The problems of transportability and the shortcomings of the standard panel truck were solved by installing the equipment in a custom body on a commercial delivery-truck chassis. This provided a short wheelbase for maneuvering in close quarters at traffic study sites and allowed a maximum of usable interior space for equipment placement and operator comfort. (A typical position for the operation of the traffic analyzer vehicle is shown on the magazine cover.)

Figure 1 shows in block diagram the instrument design of the traffic analyzer. Special vehicle detectors may be placed at any or all of

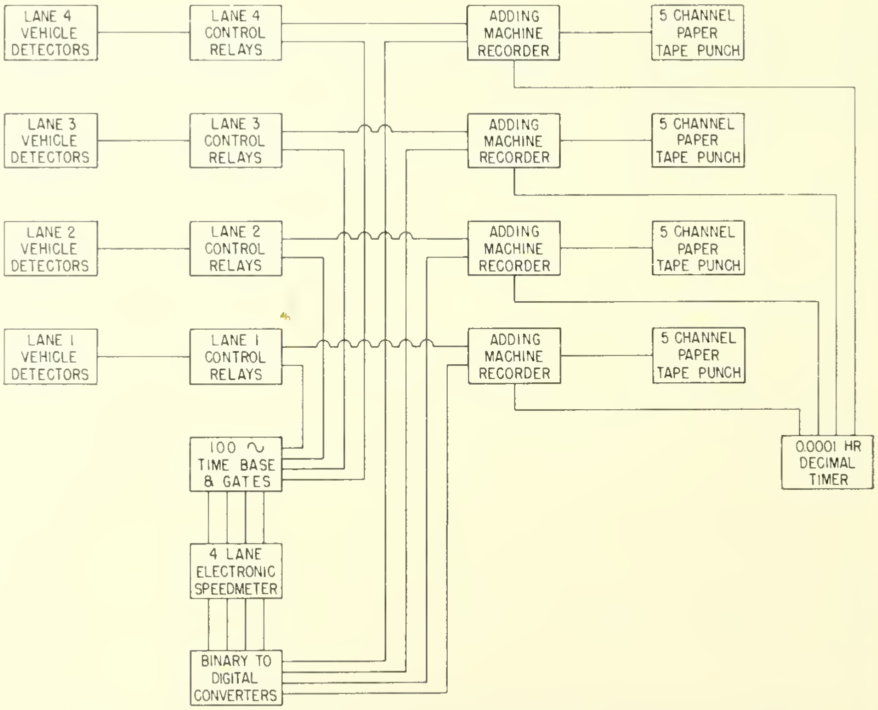

Figure 1.—A block diagram of the traffic analyzer instrument system.

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[1] This article was presented at the 30th Annual Meeting of the Institute of Traffic Engineers, Chicago, Ill., September 1960.

[2] *New techniques in traffic behavior studies*, by E. H. Holmes and S. E. Reymer, PUBLIC ROADS, vol. 21, No. 2, April 1940, pp. 29–45.

four different positions where vehicular traffic data are desired. The usual detectors consist of either two pneumatic tubes or two positive contact strips to detect speeds and a segmented detector designed and built by the Bureau to detect vehicle placement.

Figure 2 shows a typical field setup of the detectors on one roadway of a 4-lane divided highway. The speed detectors are spaced from 12 to 36 feet apart, depending on the average speed of traffic, to provide a known distance for speed timing. A 5-foot dead space is left between the active sections of the detectors in adjacent lanes to prevent lane-straddling vehicles from being recorded in more than one lane.

The segmented lateral placement detector is used in lengths of 12, 24, 36, or 48 feet, depending on the roadway width being studied. The detector is segmented into 1-foot units, each segment having an active contact of 7½ inches and a dead space of 4½ inches. The 4½-inch spacing between the active contacts permits a precise determination of vehicle placements.

The speed and placement detectors are connected by extension cables to placement relays and to several control relays for each lane of data recording.

Figure 3 shows the interior of the traffic analyzer vehicle. Along the left side, under the windows, a built-in desk provides four operating positions. At each operating position is a solenoid-operated adding machine. The recording units are commercially available 14-bank adding machines containing "read-out" units which transmit the data to the 5-channel paper tape punches mounted on the shelf above the recorders. Specially designed 115-volt a.c. solenoids are mounted over the keys of the adding machines. These solenoids receive electrical impulses and the data are automatically printed on adding machine tape and also punched in the 5-channel paper tape. Two banks of keys connected to push-button extensions are available on each adding machine so that manually coded information can be entered as desired separately for each lane of traffic.

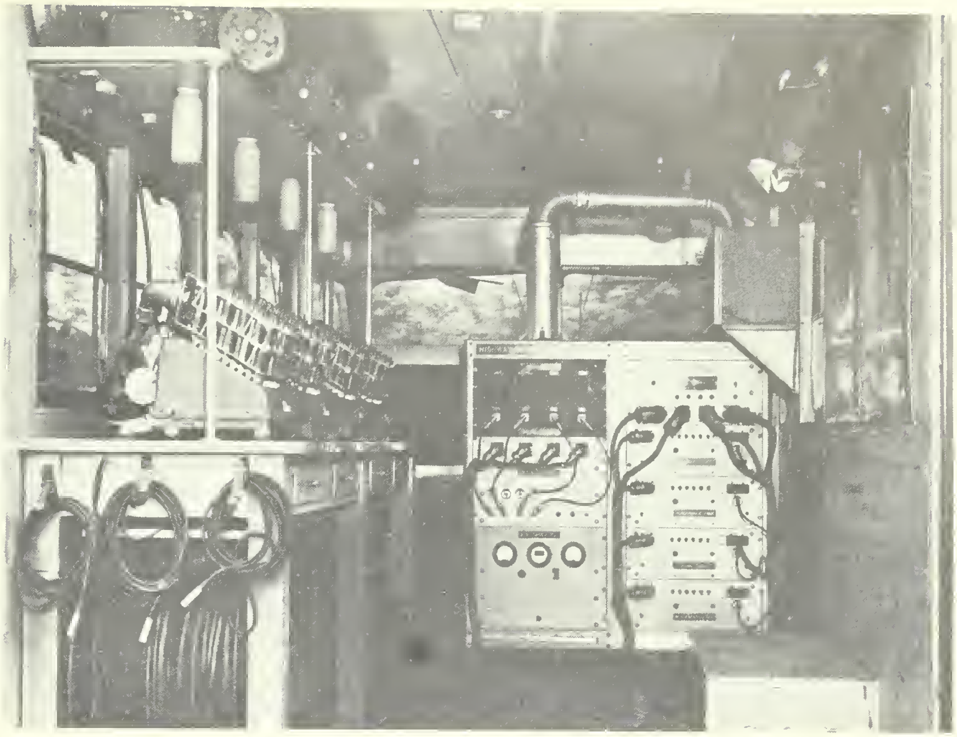

Figure 3.—An interior view of the mobile traffic analyzer vehicle.

Immediately behind the cab of the vehicle, the electronic data collecting and system-control equipment is mounted in a double relay rack cabinet. A specially fabricated cable connects the equipment in this cabinet with the four adding machines. An air conditioner, mounted above and to the right of the equipment cabinet, is necessary to dissipate the heat generated in the electronic circuitry and to cool the truck interior.

The data which are automatically recorded for each vehicle are speed, lateral placement referenced to a designated traffic lane, and precise time of day each vehicle passes the point of observation. Vehicle speed is recorded as travel time in hundredths of a second over the known distance between the speed detectors; lateral placement is recorded as a direct measurement of both the right and left wheel positions with respect to the right edge of the traffic lane, by the segmented detector; and time of day is continuously measured in intervals of one ten-thousandth of an hour. All of the data are recorded the instant that the front axle of each vehicle passes over the second speed detector.

These recorded data are subsequently taken into the office and interpreted through the use of an electronic digital computer. Vehicle speed is calculated from the travel time and the distance between detector tubes; the position of the center of the vehicle is computed from the wheel placements; and time spacings between vehicles are determined and reduced to headway distances measured in feet. The high speed computer also is used to determine the speed and the lateral and longitudinal positions of each vehicle relative to all other vehicles which are ahead, behind, or adjacent to it.

Electronic Design

The time base for the vehicle speed measurements, shown in figure 4, is a 100-cycles-per-second tuning fork. The fork and its associated amplifier and wave-shaping circuits is shown on the left. The tuning fork is electrically driven, self-starting, and is calibrated to the exact frequency of 100 cycles per second. A shutter attached to the tines of the fork, a light source above the shutter, and a photocell beneath it provide for an electronic signal output from the fork without affecting its natural frequency. The output of the time-base unit is a 100-cycle-per-second square wave of 20 volts amplitude to actuate the "one-shot" drivers of the decade scalers in the speed-timing unit. Four relay gates, one for each lane of traffic, are mounted under the time-base chassis, as shown on the right in figure 4. As they are actuated by the control relays for each lane, the gates, in turn, control the speed time pulses to be counted.

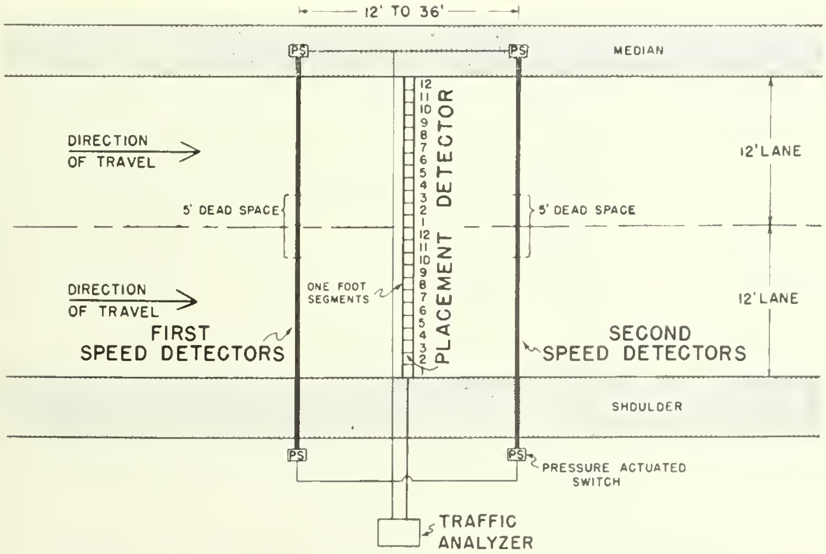

Figure 2.—A typical layout of speed and placement detectors on two lanes of a four-lane divided highway.

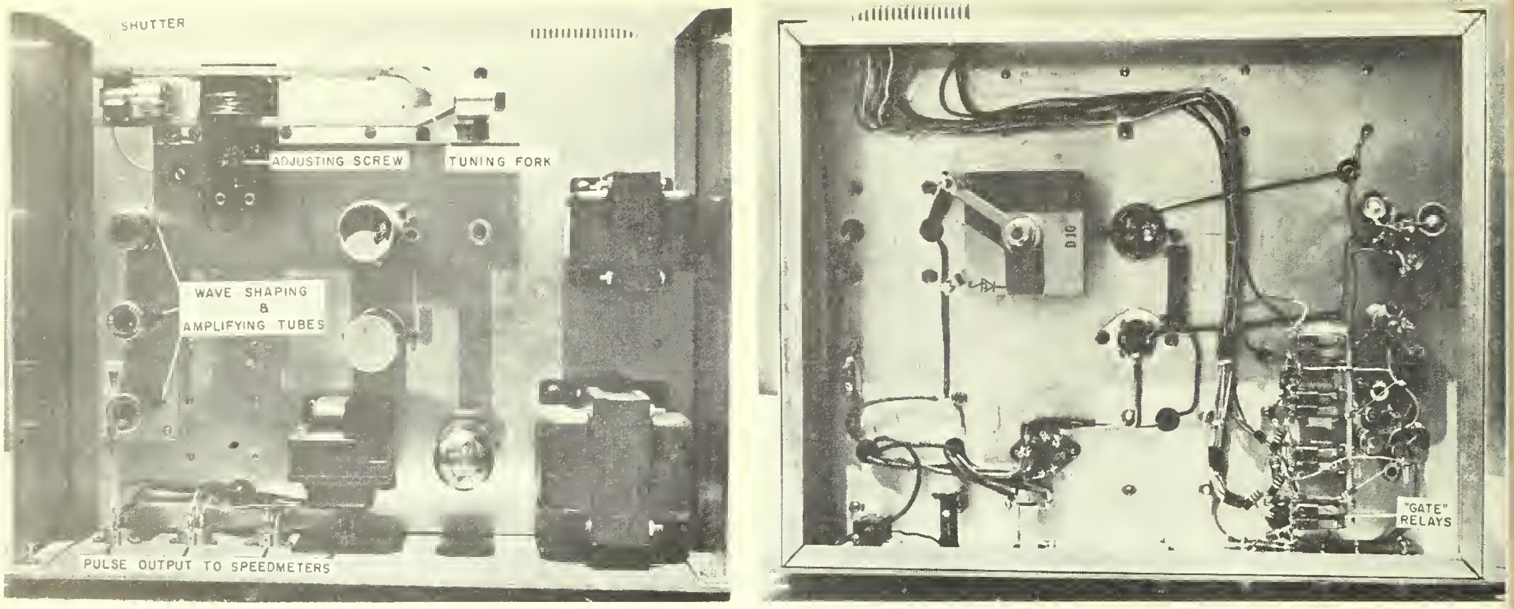

Figure 4.—Time base chassis (top view, left; bottom view, right).

The 4-lane electronic speedmeter, shown in figure 5, consists of four pairs of electronic decade scalers (on the left), each pair being capable of counting to 99. The pulse input connectors are located below each pair of scalers. As the speed-timing pulses (1/100th of a second) are received by any pair of these scalers, they are accumulated and the total held for recording. The circuitry of the speedmeter is shown on the right. The rectangular boxes in the middle of the chassis are the pulse-shaping one-shot multivibrator drivers to the decade scalers. Below these boxes are the vacuum-tube amplifiers necessary to actuate the relays in the binary to digital converter. Figure 6 is a rear view of the speedmeter, which contains the 52 vacuum tubes necessary to perform the proper functions.

The output of the decade scalers is a binary coded decimal. To record this on a digital printer a decoding relay matrix is actuated by the scaler output to provide the proper decimal number for recording. This unit is enclosed in the small panel seen immediately below the speedmeter in the interior view of the vehicle shown in figure 3.

Figure 7 shows a view of the chassis containing the placement and control relays for 1 lane of traffic. The 12 placement relays, which are actuated by the 12 contacts on the segmented placement detector, are shown in a row at the top of the figure. One power supply is needed to operate two of these relays, and the six power supplies needed are shown in the lower part of the figure. Each placement contact actuates the appropriate placement relay which, in turn, operates the proper solenoid on the adding machine. The two shift relays, shown just below the placement relays, permit automatic pickup of 3 feet of placements of the adjacent lanes to the right or left, a normal occurrence when vehicles straddle two traffic lanes. The three control relays, which are mounted on the panel, control the sequence of operation for the detection and recording of the data for the specific lane.

The time of day, accumulated in ten-thousandths of an hour, is provided by a decimal timer consisting of an 1,800-r.p.m. synchronous motor, a gear-reduction unit, and a breaker-point assembly shown to the left in figure 8. A 4-pointed cam closes an electrical contact each ten-thousandths of an hour to pulse the timing switches from the 12-volt d.c. power supply included on the chassis. These ten-thousandth-of-an-hour time units are accumulated in four 6-bank, 10-point spring-driven stepping switches. A count of 10 in one switch is carried over as a count of 1 in the next switch so that the decimal timer makes one complete timing cycle per hour. Four banks of each stepping switch provide the time information which is recorded in each of the four lanes. One bank controls the illuminated digital read-out on the timer panel, shown in figure 8 on the right, and one bank supplies the digit carry-over information to the next switch. Four push buttons on the front panel allow th timer to be preset for the start of a study.

The sequence of operation for a traffic lan being studied is initiated by a vehicle passin

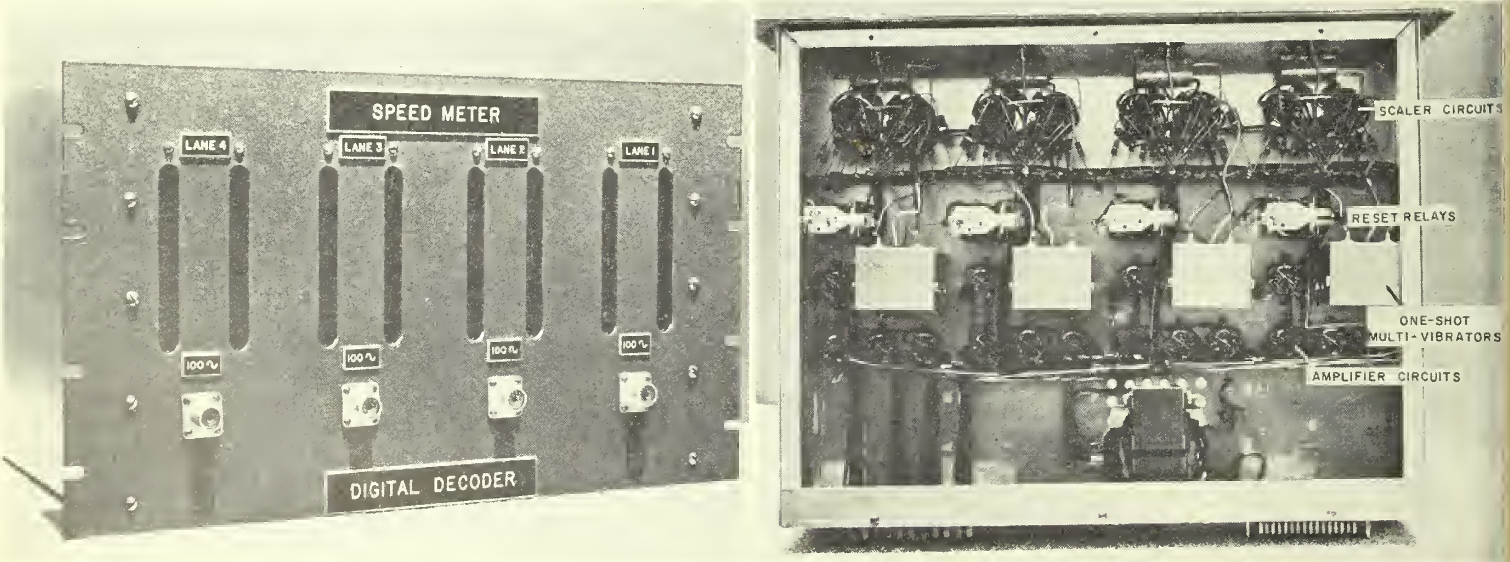

Figure 5.—Four-lane electronic speedmeter (front view, left; bottom view, right).

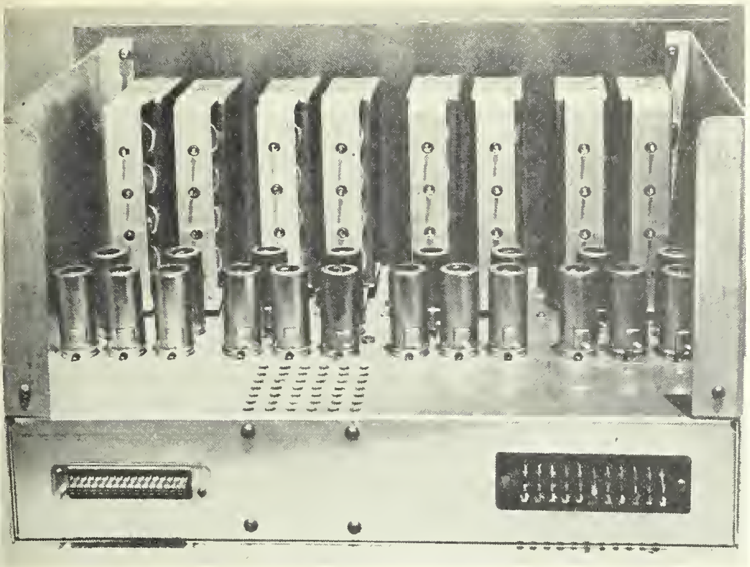

Figure 6.—Four-lane electronic speedmeter (rear view).

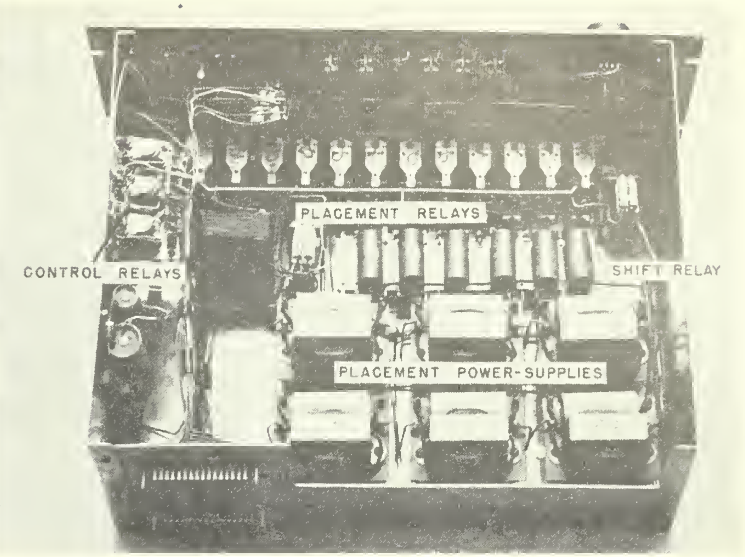

Figure 7.—Placement and control relay chassis (top view).

over the first speed detector. This electrical contact actuates a control relay which, in turn, opens the relay gate to the speedmeter and turns on the power to the placement detector for that lane. As the front wheels of the vehicle pass over the placement detector, the appropriate solenoids are actuated through the placement relays and the information is immediately entered in the adding machine recorder. When the front wheels of the vehicle pass over the second speed detector, the speedmeter gate is closed, the speed timing (in one-hundredths-of-a-second units) is entered on the recorder, the decimal timer reading (time of day in ten-thousandths of an hour) is transferred to the recorder, and a control signal causes the adding machine to cycle and print these data. During the printing cycle, the decade scalers in the speedmeter are reset to zero. The vehicle classification and any other desired information is usually set up by the observer in the two manually controlled columns in the recorder before the front axle of the vehicle closes the first positive contact detector; but it may be entered at any time before the printing cycle starts. Figure 9 shows a sample of the data recorded.

The built-in read-out to the 5-channel paper tape punch retains the necessary data after the printing cycle until the sequential operation of the punch is complete. Meanwhile, the keyboard of the adding machine will accept new data for the next recording.

Uses and Applications

The first extensive use of the original 2-lane digital recording equipment took place in 1947 on the San Francisco-Oakland Bay Bridge in California. Information was urgently needed to evaluate the driver behavior characteristics and the capacity of the bridge. Data for over 50,000 vehicles were recorded during one month at 11 locations. Because the field data were automatically printed on the adding machine tapes, it was possible to start punching machine tabulating cards after the first day of field observations. Summary tabulations showing averages and distributions of speeds, placements, and headways for all study locations were available within 2 weeks after field studies were completed. These results were very effectively used to justify specific recommendation for the improvement of traffic operations. A detailed analysis of these data also yielded invaluable criteria for lane capacity determinations discussed in the *Highway Capacity Manual*.[3]

For a 42-day period in 1948 a series of driver-behavior studies were conducted in conjunction with a study of time and gasoline consumption in motor truck operation.[4] For this purpose the equipment was used to record speeds, placements, and traffic volumes on level tangent sections, on grades, on curves, and in tunnels. The results of this study have been applied to the determination of speeds and travel time in conjunction with cost analysis of traffic operation on highways having varying horizontal and vertical alinement and in tunnels.

Since 1948, the traffic analyzer has been employed in a large number of studies in cooperation with at least 24 State highway departments, city and county traffic author-

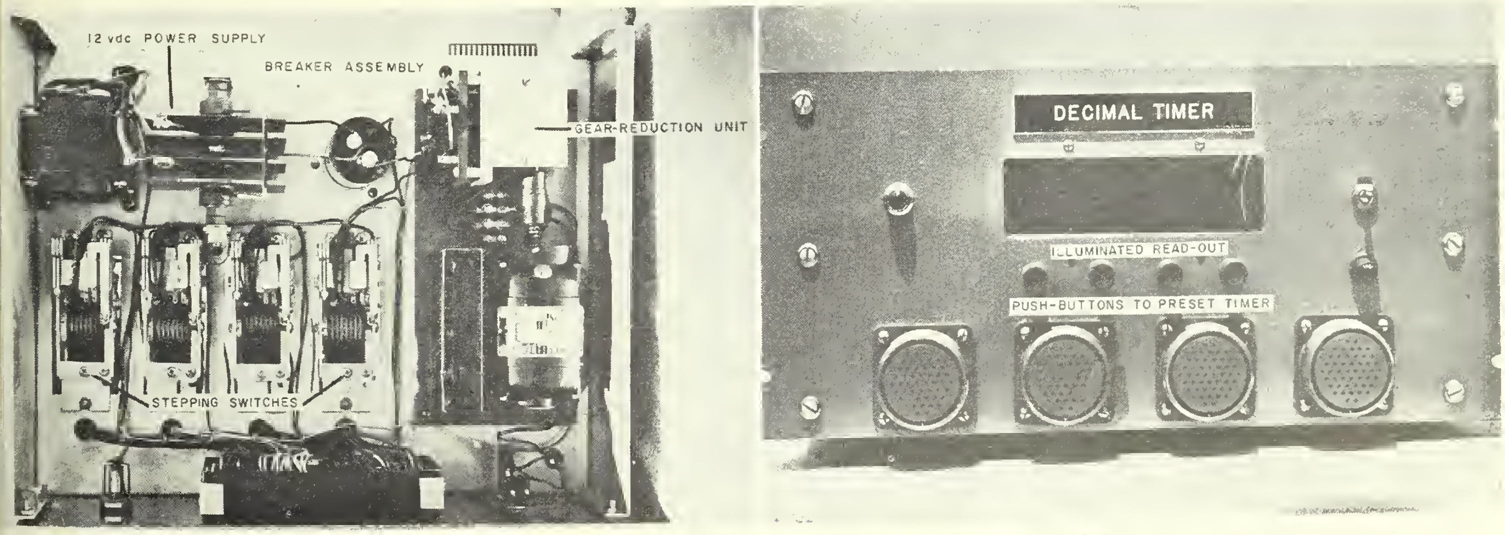

Figure 8.—Decimal timer chassis (top view, left; front view, right).

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[3] *Highway Capacity Manual*, practical applications of research, Bureau of Public Roads, 1950.

[4] *Time and gasoline consumption in motor truck operation*, as affected by the weight and power in vehicles and the rise and fall in highways, by C. C. Saal, Highway Research Board, Research Report No. 9-A, 1950.

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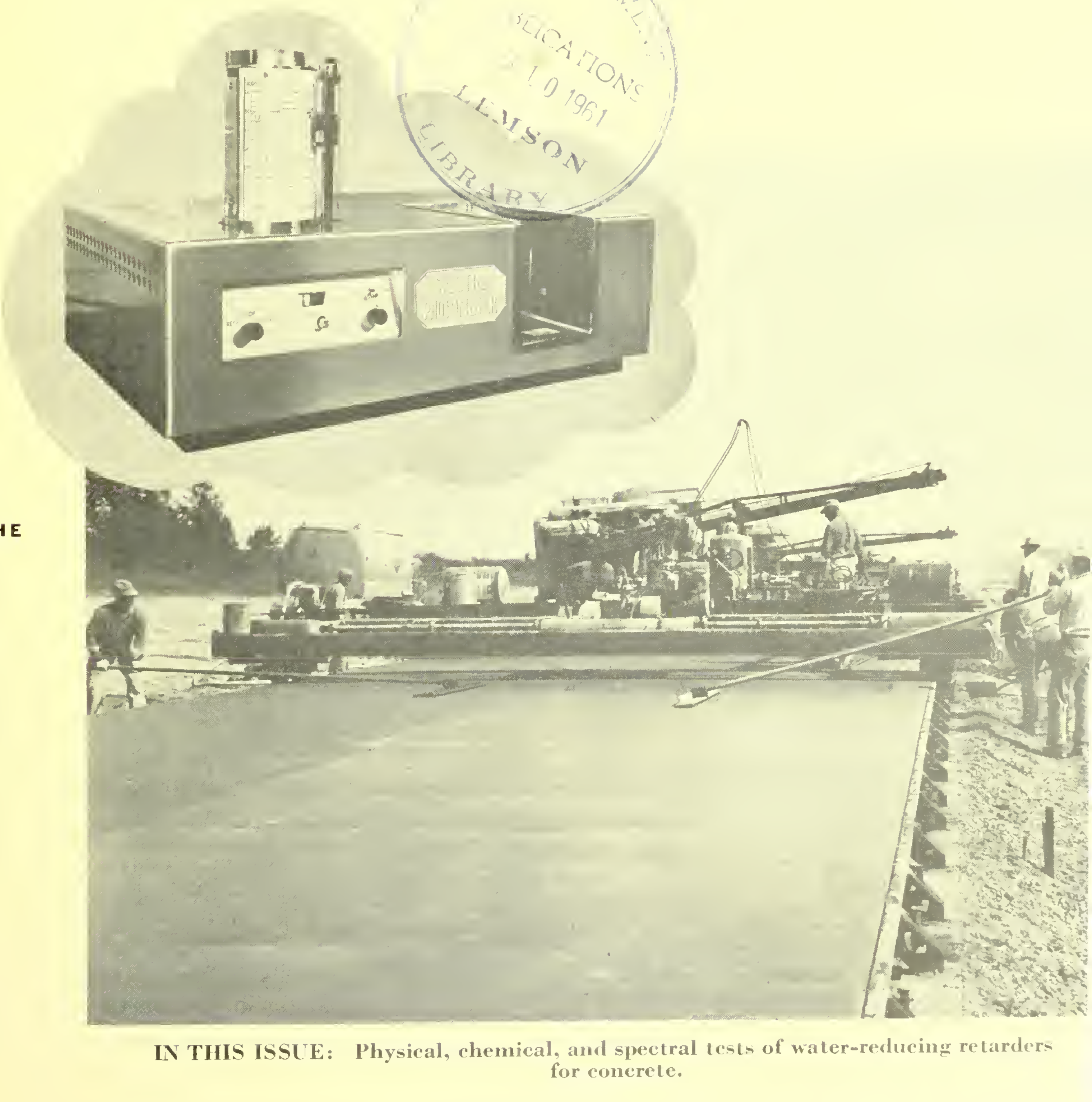

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U.S. DEPARTMENT
OF COMMERCE,
WASHINGTON

IN THIS ISSUE: Physical, chemical, and spectral tests of water-reducing retarders for concrete.

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U.S. DEPARTMENT OF COMMERCE
LUTHER H. HODGES, Secretary
BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

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Contents of this publication may be reprinted. Mention of source is requested.

A Symposium on Water-Reducing Retarders for Concrete

Within the past several years, admixtures to reduce water content and to retard set in portland cement concrete have come into prominent use in construction. With the recognition of benefits derived from and the increase in availability of commercial concrete retarders, the need for a specification for the acceptance and performance of all types of retarders has become great. Problems of acceptance and performance, however, confront the engineer, who is responsible for the selection and use of a satisfactory retarder, because of a lack of standard methods of testing retarders. It was believed that these problems would be greatly reduced by procedures for testing which would allow the engineer to make tests of the retarder to determine if it is acceptable and is uniform from batch-to-batch.

In an attempt to standardize testing methods for retarding admixtures, the Bureau of Public Roads in 1957 undertook an extensive investigation of the effects of commercial retarders on certain properties of portland cement concrete. A questionnaire was sent to all marketers of retarders known to the Bureau of Public Roads with a request for samples and information. Of the samples received, 25 samples from 11 companies were chosen for testing. The other samples were considered not to meet the Bureau's conditions for inclusion in the testing program because they were either not essentially retarders or were not available commercially at the time of receipt.

The study of commercial retarders was conducted in two phases. First, chemical, ultraviolet, and infrared spectral analyses were made to determine the composition of each retarder. Second, a study was made of the performance of the retarders when used in concrete. The results of the two phases of investigation are included in this issue. Also included is a recommended specification for water-reducing retarders.

All materials were analyzed for specific physical properties and chemical composition. One objective of the chemical and other analyses was to develop procedures by which such materials could be identified quickly to provide a basis for obtaining the necessary assurance that the composition of a product is uniform from batch-to-batch. Another objective of the analyses was to establish the chemical composition of typical commercial products to show possible relationships between chemical composition and the performance of concrete prepared with the retarders.

The term "retarder" is used for brevity in this issue to mean a water-reducing retarding admixture. The retarders studied were classified, according to their chemical constituents, into three general chemical types. For the purposes of discussion they are referred to as lignosulfonates (salts of lignosulfonic acid), organic acids (organic acids of the hydroxylated carboxylic type or their salts), and carbohydrates (reducing and nonreducing sugars and starches).

Water-Reducing Retarders for Concrete—Chemical and Spectral Analyses (p. 126) *deals with the results obtained by the various analyses of the retarders and the relative merits of each method in characterizing the composition of the admixtures. Specific conclusions based on the data are:*

1. Infrared spectral analysis offers the most promising means available for rapid identification and classification of retarders. This technique can be used to identify a material by obtaining a recorded spectral curve for each retarder. Less than 30 minutes is required for the analysis whereas a week is required by conventional methods of chemical analysis.

2. For specific construction projects, infrared analysis can be used to assure the purchaser of the uniformity of the admixture.

3. All three types of retarders could be distinguished from each other on the basis of infrared spectra. Although the lignosulfonate retarders had the same general infrared spectrum regardless of the type of salt present or the source of supply, in many instances, specific commercial lignosulfonates could be identified or differentiated by spectral differences caused by the manufacturing process or the presence of other ingredients. As to organic acids and carbohydrates, specific trade products could be distinguished from each other.

4. Ultraviolet spectra can also be used to identify lignosulfonate retarders and to obtain quantitative information as to their concentration. However, the spectra obtained are not as specific as are the infrared spectra.

5. Although useful in identifying retarder materials, conventional chemical analyses are time consuming and tedious and in many cases yield doubtful results. This was found to be especially true for the lignosulfonate and organic acid retarders.

Water-Reducing Retarders for Concrete—Physical Tests (p. 136) *reports the data of the effects of commercial retarders on certain properties of portland cement concrete. The tests reported therein cover time of retardation, reduction of water, durability, and compressive and flexural strengths at various ages. The conclusions given below apply to concrete prepared with the cement and aggregates in the stated proportions and conditions of mixing, molding, curing, and testing used in this investigation.*

1. To retard the setting time of concrete $2^1/_2$ to 3 hours, more retarder was required in some cases than that recommended by the manufacturer. This may have been due to the cement used in these tests, and indicates that acceptance tests of retarders should be made with the cement to be used in the proposed construction.

2. All retarders permitted some reduction in the amount of water required to prepare concrete with a specific consistency. This reduction was over 5 percent for 23 of the 25 retarders tested.

3. The use of retarders improved the compressive strength of concrete when the air content was less than 8 percent. This applied to concrete having either the same cement content or the same water content as was used in concrete prepared without a retarder.

4. The flexural strength of concrete in general was not reduced by the use of retarders if the air content was less than 8 percent. When the cement content of the concrete containing the retarder was the same as that for concrete without a retarder, use of the retarder generally furnished higher flexural strength at both 7 and 28 days. When the two concretes had the same water content, those containing the retarder generally had higher flexural strength at 7 days and about the same strength at 28 days.

5. As determined by freezing and thawing tests in the laboratory, the durability of concrete containing the retarder was in most cases less than that for concrete prepared without the retarder. However, the decrease in relative durability was significant in the case of only one retarder.

6. The contraction of concrete during continuous storage in laboratory air or the expansion during continuous moist storage was about the same for concrete prepared with or without a retarder.

7. An overdose of four times the normal amount of retarder generally caused a long delay in the hardening of the concrete. However, if the air content of this concrete was not over 8 percent, the compressive strength at an age of 28 days was usually greater than that of the concrete prepared without a retarder.

8. More retarder was usually required to retard the setting of concrete at a temperature of 90° F. than at 73° F. With few exceptions, the compressive strength of concrete prepared with a retarder at 90° F. was greater than that prepared at the same temperature without a retarder.

The data obtained from the study of the effects of retarders on concrete were used to prepare a specification for the acceptance and use of these retarders. The recommended specification is given on page 153. It is of interest to note that of the 25 retarders used in the test, 16 retarders met all of the requirements of the proposed specifications.

Water-Reducing Retarders for Concrete—Chemical and Spectral Analyses

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

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IN THE PAST SEVERAL YEARS, admixtures to reduce water content and retard set in portland cement concrete have come into prominent use in construction. These materials are usually complex organic products which are sold under various trade names. As yet there are no standard specifications or methods for testing retarders, but the American Society for Testing Materials (ASTM) is now considering these specifications and methods for testing such admixtures. These include among other items, requirements for the effects of water-reducing retarders on the compressive strength of the concrete, resistance to freezing and thawing, and change in volume. These tests may require a period of one year or more for completion and consequently are intended for the primary evaluation of such admixtures.

After an admixture has been found acceptable under these specifications, subsequent purchases of the same material for use on specific projects may not be subjected to extensive testing because of the time and cost involved. Consequently, the presently proposed specifications for retarding and water-reducing admixtures suggest that the purchaser obtain assurance that the admixture supplied for use on each field job or project be equivalent in composition to the original or reference admixture subjected to the exhaustive tests required by the specifications. To explore the best means of doing this, chemical analyses, as well as ultraviolet and infrared spectral analyses, were made on 25 commercial or trade-name retarders. All materials were analyzed for specific properties and chemical composition.

Types of Retarders Studied

The retarders investigated were found to be of three general chemical types: lignosulfonates; salts of organic hydroxy-carboxylic acids; and carbohydrates. A general discussion of the origin and characteristics of these major types of retarders studied is given below.

Lignosulfonates

With one exception, the lignosulfonate materials studied were derived from spent sulfite liquor obtained in the acid process of wood pulping. The single exception was derived from the Kraft (alkaline) process. Materials were supplied as the calcium, sodium, or ammonium salts, and either as a powder or in a water solution.

Lignosulfonates are considered to be polymers of high molecular weight. A single sample may contain molecules ranging in molecular weight from several hundred to 100,000 with an average molecular weight of approximately 10,000 (*1*, *2*).[3] Structurally, these materials are polymers of a substituted phenyl propane grouping. The repeating monomer unit has been represented as shown in figure 1, diagram A (*2*). The functional units of interest are hydroxyl (OH), methoxyl (OCH_3), phenyl ring, and the sulfonic group (SO_3H). In the lignosulfonate salts, a metal or ammonium cation replaces the hydrogen in the sulfonic grouping.

Many commercial lignosulfonates also contain varying amounts of reducing sugars. A typical analysis showed the following percentages of wood sugars based on total sugar content (*3*): mannose—48 percent; glucose—15 percent; xylose—15 percent; galactose—10 percent; arabinose—6 percent; fructose—under 2 percent; and sugars unaccounted for—4 percent. Much of these sugars can be removed by suitable processing techniques.

Organic hydroxy-carboxylic acids

Salts and complexes of organic hydroxy-carboxylic acids, sometimes referred to as sugar acids, are derived from the fermentation or oxidation of carbohydrates, such as dextrose, glucose, or starch. They are characterized by several hydroxyl groups and either one or two terminal carboxylic acid (COOH) groups attached to a relatively short carbon chain as illustrated in diagram B of figure 1. As retarders, they are generally supplied as metal salts in which the hydrogen in the carboxylic acid group is replaced by sodium, potassium, etc.

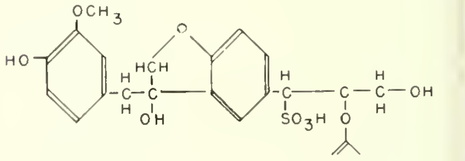
Carbohydrates

Carbohydrates, such as reducing sugars (e.g., glucose), have been used as retarders (*4*). However, non-reducing sugars, sucrose or cane sugar, are more illustrative of the type of carbohydrate evaluated in this study. The structure of sucrose is shown in diagram C of figure 1. Here again, hydroxyl groups are characteristic of the material.

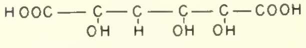
Physical Properties

The physical properties of each retarder evaluated in this study are shown in table 1. Most of the materials were powders, the rest were aqueous solutions. The color code was obtained by a visual comparison with the closest color standard available in Federal Standard No. 595. Although this Federal Standard is intended primarily for paints, it is useful for describing colors of other materials by standardized code number.

The specific gravities of the liquid samples were determined with a hydrometer. Values for pH were obtained electrometrically on liquid samples as received, and on aqueous solutions containing one percent of the solid materials. Rather low pH values, indicating significant acidity, were obtained on a complexed hydroxy-carboxylic acid retarder (No. 6) and all the ammonium lignosulfonates (Nos. 7, 10, and 17). Several calcium

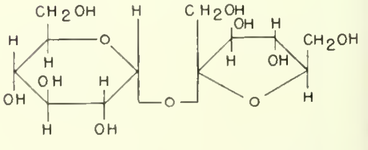


A STRUCTURAL UNIT OF LIGNOSULFONIC ACID



$$HOOC-\underset{OH}{\overset{H}{C}}-\underset{H}{\overset{OH}{C}}-\underset{OH}{\overset{H}{C}}-\underset{OH}{\overset{H}{C}}-COOH$$

B. HYDROXY—CARBOXYLIC ACID



C. CARBOHYDRATE (SUCROSE)

Figure 1.—Typical chemical structures of retarders studied.

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[1] Presented at the 40th Annual Meeting of the Highway Research Board, Washington, D.C., January 1961.

[2] Acknowledgment is made to Raymond Cherwinski and L. B. Hayes, Analytical Chemists, for their assistance in carrying out the chemical and ultraviolet analyses.

[3] Italic numbers in parentheses refer to a list of references on page 135.

Table 1.—Physical properties of retarders

Retarder No.	Form	Color		Odor	Specific gravity, 77° F./77° F.[2]	pH [3]	Solubility in—[4]			Foaming properties, milliliters of foam after—[5]				Unusual characteristics of 1-percent aqueous solution
		Observed	Code [1]				Water	Alcohol benzene chloroform	Dimethyl sulfoxide	1 minute	2 minutes	5 minutes	15 minutes	
1	Liquid	None	------	Phenolic	1.143	7.4	----	--------	------	0	0	0	0	
2	Solid	White	37778	Woody	--------	7.8	I	I	I	0	0	0	0	
3	do	Olive	30118	do	--------	7.2	S	I	I	10	5	3.5	1	
4	Liquid	Amber [6]	[6] 23655	Rancid	1.182	6.8	----	--------	------	0	0	0	0	U. V. fluorescence.
5	do	Wine	20109	Pungent	1.148	7.1	----	--------	------	0	0	0	0	
6	do	Amber	33538	None	1.178	2.8	----	--------	------	0	0	0	0	
7	Solid	Brown	10091	Woody	--------	3.3	PS	I	I	10	7	4	1.5	Mold growth.
8	do	Pink	30313	None	--------	7.6	I	I	I	0	0	0	0	
9	Liquid	Dark brown	10032	Rancid	1.196	8.2	----	--------	------	0	0	0	0	
10	Solid	Medium brown	10091	Woody	--------	3.2	S	I	I	2	2	1.5	1	Mold growth.
11	do	Light brown	20400	do	--------	6.8	S	I	I	4	2	0	0	Do.
12	do	Mustard	30266	do	--------	6.7	S	I	PS	0	0	0	0	
13	do	Medium brown	30117	do	--------	9.4	PS	I	I	1	.5	0	0	
14	do	Tan	31643	do	--------	4.6	S	I	I	5	4	3	1.5	
15	do	Olive	30009	do	--------	7.4	S	I	I	11	8	4.5	2	
16	do	Tan	31643	do	--------	7.3	PS	I	PS	9	6	2.5	1	
17	do	Light brown	20400	do	--------	3.8	S	I	S	1.5	1	.5	0	Mold growth.
18	do	Tan	31643	do	--------	7.4	S	I	I	3.5	2	1	.5	
19	do	Light brown	20400	do	--------	6.6	S	I	I	.5	0	0	0	
20	Liquid	Dark brown	10032	do	1.155	4.8	----	--------	------	12	8	4	1	
21	do	do	10032	do	1.147	4.7	----	--------	------	10	7	3	.5	
22	Solid	Dark brown	30045	do	--------	8.8	S	I	I	5	2.5	1	.5	
23	do	Medium brown	10091	do	--------	9.5	S	I	I	0	0	0	0	
24	do	Tan	31643	do	--------	7.8	PS	I	I	1	1	.5	0	
25	do	do	31643	do	--------	4.6	PS	I	I	7	7	7	7	

[1] Federal Standard No. 595 (by visual matching).

[2] Hydrometer method on liquid samples as received.

[3] Electrometric method on liquid samples as received, and on solid samples in 1-percent water solution.

[4] At room temperature: I=insoluble (less than 0.01 g. in 10 ml.); PS=partially soluble (between 0.01 and 1 g. in 10 ml.); S=soluble (more than 1 g. in 10 ml.).

[5] 20 ml. of 1-percent aqueous solution in 50 ml. stoppered graduate. Inverted 30 times. Readings then noted at indicated time intervals. (Modified from McCutcheon, J. W., *Synthetic Detergents*, 1950, p. 79.)

[6] By transmitted light. By reflected light, color had a greenish fluorescence (equivalent to Code 14187).

lignosulfonates (Nos. 14, 20, 21, and 25) had moderately low pH values, possibly as a result of incomplete neutralization of the sulfonic acid groups during processing.

The apparent insolubility of samples Nos. 2 and 8 in water was caused by the large amount of inert filler in each material. Ultraviolet fluorescence was observed on a 1-percent aqueous solution of sample No. 4, most likely because of a fluorescent dye used by the producer to characterize his product. As noted in table 1, a mold growth developed in several of the 1-percent aqueous solutions after standing for about one week. Only retarder No. 25 produced a lasting foam in the foaming test. This was apparently a result of some synthetic detergent which has been incorporated into the admixture.

Qualitative Chemical Test Results

Table 2 shows the results of qualitative chemical tests used to identify and classify each material for further evaluation. References to the methods used are given in the footnotes to the

Table 2.—Qualitative tests on retarders [1]

Retarder No.	Sulfonated organics (Basic fuchsin test) [2]			Lignosulfonate (Proctor-Hirst test) [3]	Chloride (AgNO$_3$ test)	Sulfate (BaCl$_2$ test)	Carbonate (acid effervescence)	Calcium (oxalate test)	Sodium (flame phot.)	Potassium (flame phot.)	Nitrogen		Hydroxy acids (dinaphthol test) [5]	Complexing agents (CuSO$_4$+NaOH test) [6]	Melting point of phenylhydrazine derivative, °C. [6]
	Original material	Alcohol-insoluble fraction	Alcohol-soluble fraction								Ammoniacal [4]	Albuminoid [4]			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1 [7] [8]	N	-----	-----	N	N	N	N	N	T	N	-----	-----	----	P (light blue)	
2	P	P	N	T	N	N	P	N	T	N	N	N	----	----	No derivative.
3	P	P	N	P	N	Q	N	P	P	T	N	N	----	----	
4	N	-----	-----	N	N	N	N	N	P	T	-----	-----	P (green)	P (light blue)	188 to 189.
5 [9]	N	-----	-----	N	P	N	N	N	T	T	N	P	P (blue-green)	P (medium blue)	189 to 191.
6 [10]	N	-----	-----	N	N	N	N	N	T	N	-----	-----	----	P (light blue)	No derivative.
7	P	P	N	P	N	Q	N	N	T	T	P	P	----	----	
8	N	-----	-----	N	N	N	N	P	P	T	-----	-----	P (light green)	P (light blue)	No derivative.
9	N	-----	-----	N	P	N	N	N	P	P	-----	-----	P (green)	P (blue-green)	180 to 182.
10	P	P	N	P	N	Q	N	N	T	T	P	P	----	----	
11	P	P	N	P	P	P	N	P	P	P	N	N	----	----	
12	P	P	N	P	N	N	N	P	P	T	N	N	----	----	
13	P	P	N	P	N	N	P	T	P	T	N	N	----	----	
14	P	P	N	P	N	P	N	P	P	T	N	N	----	----	
15	P	P	N	P	N	P	N	N	P	T	N	N	----	----	
16	P	P	N	P	N	P	N	P	P	T	N	N	----	----	
17	P	P	N	P	N	Q	N	N	T	T	P	P	----	----	
18 [11]	P	P	N	P	N	N	N	P	P	T	N	N	----	P (light green)	No derivative.
19	P	P	N	P	N	Q	N	N	P	P	N	N	----	----	
20	P	P	N	P	N	N	N	P	T	T	N	N	----	N	
21	P	P	N	P	N	N	N	P	T	T	N	N	----	N	
22	P	P	N	P	N	Q	N	N	P	N	N	N	----	----	
23	P	P	N	P	N	Q	N	N	P	T	P	P	----	----	
24	P	P	N	P	P	T	N	P	P	P	P	P	----	----	
25	P	P	P	P	N	P	N	P	P	T	N	N	----	----	

[1] Code: P=positive indication; N=negative indication; T=trace indicated; Q=questionable indication.

[2] Modified from Wallin, Anal. Chem., vol. 22, 1950, pp. 616-7.

[3] Jour. ALCA, vol. 51, No. 7, 1956, pp. 353-76.

[4] PUBLIC ROADS, vol. 27, No. 12, 1954, p. 268.

[5] F. Feigl, Qualitative Analysis by Spot Tests, 3d ed., 1946.

[6] Test procedure given in text.

[7] A positive indication of phenols was obtained by the Millon test (Feigl, Qualitative Analysis by Spot Tests, 3d ed., 1946, p. 330).

[8] A positive indication of sucrose was obtained by the alpha-naphthol test (Griffin, R. C., Technical Methods of Analysis, 2d ed., 1927, p. 567).

[9] A positive indication of triethanolamine was obtained by the Kraut test (*Official Methods of Analysis*, A.O.A.C., 7th ed. 1950, p. 609).

[10] Positive indications of zinc (sulfide test), and boron (flame and turmeric tests) were obtained.

[11] A negative test for triethanolamine was obtained by means of the Kraut test (see footnote 9).

table. Positive test results are shown by "P", negative results by "N", trace quantities by "T", and questionable results by "Q".

Sulfonated organic material was identified with basic fuchsin. This reagent reacts with sulfonates in acid solution to form a magenta-colored complex which can be extracted with chloroform. The results are given in column 2. The method has been described as a quantitative spectrophotometric determination of sulfonated and sulfated organic materials (*5*). When a positive result was obtained on the original material, additional tests were made on the alcohol-insoluble and alcohol-soluble fractions of the retarder. A positive result on the alcohol-insoluble fraction suggests the presence of lignosulfonates, since lignosulfonates are insoluble in alcohol. A positive test result on the alcohol-soluble fraction, as for retarder No. 25, suggests the presence of a sulfonated or sulfated synthetic detergent. Consequently, the result on sample No. 25 reinforces the findings of the foaming test, that this material contained a synthetic detergent.

A more specific indication of lignosulfonate is given by the Proctor-Hirst test. In this test, an acidified clear extract is treated with aniline. A resultant cloudiness is indicative of lignosulfonate. The test is considered to be quite reliable (*6*). In every case, these results are in agreement with the findings from the basic fuchsin test.

Columns 6 through 11 in table 2 show the the results of tests for chloride, sulfate, carbonate, calcium, sodium, and potassium using standard procedures. Positive test results for chloride and sulfate are not entirely reliable in the presence of some organics, such as were found in some retarders, because of the possible precipitation of organic salts in the presence of silver and barium reagents.

Test results for both ammoniacal and albuminoid nitrogen are shown in columns 12 and 13. In the test for ammoniacal nitrogen, an alkaline solution of the sample was boiled and the vapor tested with red litmus paper for volatile ammonia. A blue color indicated the presence of an ammonium salt. Following the removal of volatile ammonia, permanganate was added and the test repeated to detect albuminoid nitrogen. Positive results suggest the presence of amines and similar nitrogenous compounds.

Zinc was detected in sample No. 6 by precipitation as a white sulfide, and boron by flame tests and turmeric paper. In sample No. 1, a phenolic odor was detected which was attributed to a phenol-type material. This phenolic material was confirmed by the Millon test. This material probably serves as a fungicide. The same retarder also gave a positive test result for sucrose. However, substances similar to sucrose may give positive results with alpha-naphthol reagent, so that these results were not conclusive.

The results of the dinaphthol test which has been used to identify many hydroxy-carboxylic acids are shown in column 14. Acids such as tartaric, malic, tartronic, glucoronic, and gluconic reportedly give a green fluorescence (*7*). Where a positive test result was obtained, the nature of the color which developed is shown in parentheses.

Test results for complexing agents are shown in column 15. Although not specific, the test may be used to detect such complexing agents as triethanolamine, sugars, hydroxy-carboxylic acids, etc.[4] Where positive results are shown, the color of the copper complex formed is also given.

A specific check for triethanolamine was made by means of the Kraut test. Sample No. 5 gave a positive indication of the presence of this compound, while sample No. 18 gave a negative test for triethanolamine.

Column 16 shows the melting points of phenylhydrazine derivatives of several retarders. Ordinarily, such derivatives are obtained with carboxylic acids, and the melting points of these derivatives have been used for qualitative analyses.[5] This test was suggested by one producer of retarders; however, in this study it was found that duplicate determinations failed to give reasonably reproducible results.

On the basis of the above qualitative tests, the following summarization was possible. Retarder No. 1 contained a sugar and a phenolic additive. Five samples were found to be the following derivatives of hydroxy-carboxylic acids: Nos. 4, 8, and 9 appeared to be metal salts; No. 6 a zinc borate complex; and No. 5 a triethanolamine salt. The other materials were lignosulfonate salts of various types.

Quantitative Chemical Analyses

Inorganic constituents

Table 3 gives the results of the analyses for inorganic constituents in each retarder sample. Moisture or water was determined by oven loss at 105° C. except for sample No. 5. This material had a tendency to decompose or volatilize at that temperature, and therefore its moisture content was determined by heating at 50° C. under vacuum.

Total ash content was determined by ignition at 600° C., and HCl insoluble by treating the ash with hydrochloric acid (1:5). The acid-soluble ash constituents were then determined by conventional methods of analysis and reported as the oxides. The alkalies, Na_2O and K_2O, were determined by flame photometry, and zinc by ferrocyanide titration with an external indicator. Carbon dioxide was determined by wet evolution with hydrochloric acid and chlorides by the Mohr titration. Boron was analyzed by distillation with methyl alcohol followed by titration using an ASTM standard procedure. Total sulfur was obtained by wet oxidation followed by precipitation as barium sulfate.

Summary of inorganic results

Retarders Nos. 1, 5, 7, 10, and 17 appeared to be composed almost exclusively of organic material that was completely volatilized at 600° C. Retarder No. 2 contained limestone or dolomitic limestone along with zinc, perhaps as zinc oxide. Samples Nos. 4 and 9 were composed of sodium and potassium salts, respectively, of carboxylic acids. Sample No. 6 contained zinc and boron, most likely in the oxide form. Retarder No. 8 contained a large amount of insoluble siliceous material and iron oxide. Retarders Nos. 11 and 24 contained calcium chloride and No. 13 contained a substantial amount of sodium carbonate.

Organic constituents

The organic constituents determined in each retarder are shown in table 4. References to the methods used are given in the footnotes. The volatile matter at 600° C. is an approximate measure of total organic material present. Methoxyl was determined by the Zeisel method.

Lignosulfonates were determined by a recommended cinchonine procedure. It appears from the results shown, as well as the results of duplicate determinations, that the cinchonine procedure yielded erratic results. Lignin content was also calculated from the methoxyl values as given by the Technical Association of the Pulp and Paper Industry (TAPPI method T629, m-53). While this procedure was empirical, the results appeared to be more realistic than those obtained by the cinchonine method.

A colorimetric procedure was used to determine total carbohydrates by the anthrone method (*8*), and reducing sugars were determined by the Somogyi-iodometric method, after precipitation of non-carbohydrates with basic lead acetate. In both cases the constituent was calculated as glucose.

Sucrose was determined by the AOAC (Association of Official Agricultural Chemists) official chemical procedure by direct weighing of cuprous oxide. The anionic sulfonated detergent was determined colorimetrically with basic fuchsin after first extracting the detergent with alcohol. The method used was similar to that prescribed by Wallin (*5*) except that visual estimation was made in Nessler tubes because of the staining effect by the reagent on spectrophotometric cells.

Phenol was determined from the spectrophotometric absorption at 270 millimicrons, using a calibration curve of known concentrations of phenol plotted against absorbance.

Total nitrogen was obtained by the Kjeldahl distillation procedure, and ammoniacal nitrogen was determined by distillation from an alkaline solution. After removing ammoniacal nitrogen, permanganate was added and albuminoid nitrogen obtained by distillation. Fixed nitrogen was calculated by difference.

Classification and probable composition

Based on the results of chemical analysis, the classification and probable composition of each retarder are given in table 5. The majority of the retarders analyzed were found to be ammonium, sodium, or calcium salts of lignosulfonic acid. The approximate amount of lignosulfonate salt shown was obtained by empirical calculations based on methoxyl values. In two samples, Nos. 3 and 15, the empirical calculation gave unreasonably high results because of high methoxyl contents. Consequently, the values shown for these samples were obtained by difference and therefore represented the maximum amounts of lignosulfonate possible. Carbohydrates, both nonreducing and reducing-sugar types, were calculated as glucose.

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[4] Details of the test procedure are as follows: To 10 ml. of a 1-percent solution of the admixture, add 1 ml. of 10-percent NaOH, mix, and add 1 ml. of 3-percent $CuSO_4$. Mix and note whether a soluble colored copper complex is formed.

[5] Details of the test procedure are as follows: To a test tube (35 ml., 20 x 150 mm.) add 1 ml. of a 50-percent aqueous solution of active ingredient, add 4 ml. H_2O, 0.7 ml. of glacial acetic acid and 1.0 ml. phenyl hydrazine. Fit with reflux tube approximately 8 inches long, heat in boiling water and reflux 3 hours. Filter while hot, collect filtrate, cool and let crystallize from 2 hours to overnight. Collect crystals by filtration, wash with 5 ml. cold water, then 5 ml. alcohol. Dry at room temperature and determine melting point.

Only one lignosulfonate retarder, No. 25, was found to contain a sulfonated synthetic detergent, which was possibly a sodium alkyl aryl sulfonate. Two of the lignosulfonates, Nos. 11 and 24, contained substantial amounts of calcium chloride (9.8 and 25 percent, respectively). Retarder No. 2 contained 80–90 percent of dolomitic limestone and about 7 percent zinc oxide. Retarder No. 13 contained approximately 29 percent sodium carbonate.

The derivatives of hydroxy-carboxylic acids were found to be metallic salts—sodium, calcium, and potassium; triethanolamine salt; and zinc borate complex. These materials contained little or no carbohydrates. Retarder No. 8, a solid material, contained iron oxide and siliceous filler. The organic carboxylic material in each retarder was estimated by difference.

Only one retarder, No. 1, was found to be a carbohydrate. This was essentially a nonreducing sugar (sucrose). It contained some phenol to prevent fermentation or mold growth.

It is quite possible that these retarders may contain minor amounts of other organic substances not discussed here, but no effort was made in this study to identify all such minor constituents.

Table 3.—Inorganic constituents in retarders, by percent of constituent

Retarder No.	Moisture or water, at 105° C.[1]	Analyses on dry solids basis[2]													
		Total ash at 600° C.	HCl (1:5) Insoluble	SiO_2	R_2O_3 [3]	Fe_2O_3	CaO	MgO	Na_2O	K_2O	SO_3	ZnO[4]	CO_2, wet evolution method	Chloride as $CaCl_2$	B_2O_3 [5]	Total sulfur, as SO_3 [6]
1	66.7	0.2	------	------	------	------	------	------	------	------	------	------	------	------	------	------
2	.4	97.2	5.6	Neg.	0.8	------	34.3	5.1	Neg.	Neg.	0.9	6.7	37.8	------	------	0.6
3	6.8	17.7	.1	0.2	.4	------	3.2	1.0	5.2	0.1	6.3	------	------	0.0	------	9.8
4	68.2	24.8	Neg.	Neg.	Neg.	------	Neg.	Neg.	12.6	.2	Neg.	------	------	------	------	------
5	[7] 57.1	.4	------	------	------	------	------	------	------	------	------	------	------	------	------	.0
6	72.5	32.5	Neg.	Neg.	------	------	------	------	Neg.	Neg.	Neg.	15.6	------	------	11.9	------
7	4.5	1.3	.1	Neg.	Neg.	------	.3	.2	.4	.2	Neg.	------	------	------	------	14.3
8	.7	90.0	[8] 67.5	2.2	14.0	6.7	5.2	2.0	.9	.1	Neg.	------	------	------	------	------
9	66.5	32.4	Neg.	.8	4.0	------	Neg.	Neg.	.5	16.5	Neg.	------	------	------	------	------
10	5.1	.7	Neg.	.2	.1	------	Neg.	Neg.	Neg.	Neg.	Neg.	------	------	------	------	14.0
11	6.2	28.2	.6	Neg.	Neg.	------	11.4	.2	2.1	.3	6.7	------	------	9.8	------	13.2
12	4.4	21.2	.2	Neg.	.2	------	10.8	.7	.2	.2	6.0	------	------	.0	------	11.4
13	8.8	47.0	.2	.1	.6	------	3.1	.6	16.3	.1	10.1	------	12.6	------	------	9.6
14	6.5	15.2	.1	Neg.	.1	------	4.6	.4	.1	.3	3.3	------	------	------	------	9.2
15	7.4	23.7	.3	.2	Neg.	------	.6	1.1	9.2	.1	9.0	------	------	------	------	12.0
16	5.3	16.4	2	Neg.	Neg.	------	6.7	.5	.7	.4	7.1	------	------	.0	------	13.5
17	7.6	.8	Neg.	Neg.	Neg.	------	Neg.	Neg.	.1	.3	2	------	------	------	------	13.5
18	7.2	9.4	.1	Neg.	.6	------	5.2	Neg.	Neg.	Neg.	3.2	------	------	------	------	11.8
19	5.4	25.4	.1	.3	.1	------	Neg.	Neg.	9.6	.1	13.6	------	------	------	------	16.9
20	69.0	22.5	Neg.	.2	Neg.	------	9.5	Neg.	Neg.	Neg.	10.8	------	------	------	------	13.1
21	70.3	22.3	Neg.	.2	Neg.	------	9.1	Neg.	Neg.	Neg.	11.9	------	------	------	------	15.6
22	8.5	45.2	.3	Neg.	.3	------	Neg.	Neg.	20.0	.1	22.2	------	------	------	------	27.2
23	7.2	34.7	2	Neg.	Neg.	------	6	.1	12.8	.3	10.2	------	------	------	------	7.6
24	4.0	44.8	6	.1	Neg.	------	17.7	.3	3.2	.4	9.5	------	------	25.0	------	16.0
25	6.5	17.2	.1	Neg.	.1	------	6.9	.7	.7	.1	6.7	------	------	------	------	15.6

[1] Loss at 105° C., calculated on basis of material as received.
[2] Neg. = Negligible.
[3] Fe_2O_3 plus Al_2O_3.
[4] Volumetric determination with ferrocyanide.
[5] Distillation method, modified from method C 169–53, sec. 18; ASTM Standards Part. 3, 1955, p. 907.
[6] TAPPI, T 629-m53.
[7] Loss at 50° C., vacuum.
[8] Mainly siliceous matter.

Table 4.—Organic constituents, in percentages by weight of dry solids

Retarder No.	Volatile matter at 600° C.	Methoxyl[1]	Lignosulfonate (cinchonine method)[2]	Lignin, calculated from methoxyl[3]	Total carbohydrates, as glucose[4]	Reducing sugars, as glucose[5]	Sucrose (AOAC method)[6]	Anionic sulfonated synthetic detergent[7]	Phenol[8]	Nitrogen			
										Total (as N)[1]	Ammoniacal (as NH_3)[1]	Albuminoid (as N)[9]	Fixed (as N)[10]
1	99.8	--------	--------	--------	--------	0.0	96.3	--------	1.4	--------	--------	--------	--------
2	2.8	0.2	1.3	1.7	0.7	6	--------	--------	--------	--------	--------	--------	--------
3	82.3	11.8	69.2	92.2	4.0	8	--------	0.0	--------	--------	--------	--------	--------
4	75.2	--------	--------	--------	.4	.2	--------	--------	--------	--------	--------	--------	--------
5	99.6	--------	--------	--------	.9	5	--------	--------	--------	3.9	0.1	2.0	3.8
6	67.5	--------	--------	--------	.6	01	--------	--------	--------	--------	--------	--------	--------
7	98.7	8.2	109.6	64.2	5.9	2.9	--------	--------	--------	3.2	2.0	--------	1.7
8	10.0	--------	--------	--------	1.0	.05	--------	--------	--------	--------	--------	--------	--------
9	67.6	--------	--------	--------	.4	1	--------	--------	--------	--------	--------	--------	--------
10	99.3	8.4	43.7	65.2	5.4	3.5	--------	--------	--------	3.4	1.4	--------	2.2
11	71.8	6.4	100.6	49.8	5.2	1.5	--------	--------	--------	--------	--------	--------	--------
12	78.8	7.2	103.8	56.2	4.7	2.7	--------	--------	--------	--------	--------	--------	--------
13	53.0	7.1	123.4	55.5	.8	3	--------	--------	--------	--------	--------	--------	--------
14	84.8	6.6	110.2	51.9	9.1	5.1	--------	--------	--------	--------	--------	--------	--------
15	76.3	11.4	104.4	88.6	2.5	.3	--------	--------	--------	--------	--------	--------	--------
16	83.6	7.5	96.9	58.8	7.6	3.8	--------	--------	--------	--------	--------	--------	--------
17	99.2	8.2	98.1	63.9	12.2	8.1	--------	--------	--------	3.2	2.7	--------	1.1
18	90.6	6.9	114.6	53.7	8.6	3.0	--------	--------	--------	1.8	.0	.5	1.8
19	74.6	7.1	92.8	54.7	8.9	4.8	--------	--------	--------	--------	--------	--------	--------
20	77.5	6.2	93.0	48.7	5.4	1.3	--------	--------	--------	--------	--------	--------	--------
21	77.7	7.3	117.6	56.9	6.8	2.4	--------	--------	--------	--------	--------	--------	--------
22	54.8	5.4	57.5	42.3	.6	.02	--------	--------	--------	--------	--------	--------	--------
23	65.3	6.9	65.5	53.7	2.3	.3	--------	--------	--------	.9	.0	--------	.9
24	55.2	2.8	93.5	21.9	1.4	.3	--------	--------	--------	1.0	.2	.6	.8
25	82.8	8.6	103.5	67.2	10.0	4.7	--------	1.1	--------	--------	--------	--------	--------

[1] PUBLIC ROADS, vol. 27, No. 12, p. 268, 1954.
[2] Jour. ALCA, vol. 51, No. 7, p. 353, 1956.
[3] TAPPI, T629-m53; methoxyl divided by 0.128.
[4] Anthrone method; Morris, Science, vol. 107, 1948, p. 254–5.
[5] Somogyi iodometric volumetric method, Jour. Biol. Chem., vol. 160, No. 1, 1945, p. 61.
[6] AOAC method 29.29.
[7] Colorimetric method (basic fuchsin), Wallin, Anal. Chem., vol. 22, 1950, p. 616.
[8] By spectrophotometric absorption at 270 millimicrons.
[9] AOAC method 2.37.
[10] Calculated from: total nitrogen − ammoniacal nitrogen.

Table 5.—Classification and probable composition of retarders, percent of constituent on dry solids basis [1]

Retarder type	Retarder No.	Water [2]	Lignosulfonate salt [3]	Carbohydrates [4]		Calcium chloride	Zinc oxide	Iron oxide and alumina	Organic carboxylic material [7]	Miscellaneous constituents (percent)
				Non-reducing types [5]	Reducing sugars [6]					
Lignosulfonates:										
Ammonium salts	7	4.5	76	3.0	2.9	------	------	------	--------	
	10	5.1	77	1.9	3.5	------	------	------	--------	
	17	7.6	76	4.1	8.1	------	------	------	--------	
Sodium salts	15	7.4	[9] 94	2.2	.3	------	------	------	--------	
	19	5.4	66	4.1	4.8	------	------	------	--------	
	[8] 22	8.5	51	.6	.02	------	------	------	--------	
	23	7.2	64	2.0	.3	------	------	------	--------	
Calcium salts	2	.4	2	.1	.6	------	6.7	------	--------	80–90 dolomitic limestone.
	3	6.8	[9] 93	3.2	.8	0.0	------	------	--------	0.0 synthetic detergent.
	11	6.2	59	3.7	1.5	9.8	------	------	--------	
	12	4.4	67	2.0	2.7	.0	------	------	--------	
	13	8.8	66	.5	.3	------	------	------	--------	29 sodium carbonate.
	14	6.5	62	4.0	5.1	------	------	------	--------	
	16	5.3	70	3.8	3.8	.0	------	------	--------	
	18	7.2	64	5.6	3.0	------	------	------	--------	
	20	69.0	58	4.1	1.3	------	------	------	--------	
	21	70.3	68	4.4	2.4	------	------	------	--------	
	24	4.0	26	1.1	.3	25.0	------	------	--------	
	25	6.5	80	5.3	4.7	------	15.6	------	--------	1.1 synthetic detergent.[11]
Organic acids:										
Metal salts:										
Sodium (9.3 percent Na)	4	68.2	------	.2	.2	------	------	------	90	
Calcium (3.7 percent Ca)	8	.7	------	.9	.1	------	------	14	10	71 inert siliceous matter.
Potassium (13.7 percent K)	9	66.5	------	.3	.1	------	------	------	86	
Triethanolamine salt	5	57.1	------	.5	.5	------	------	------	57	42 triethanolamine.[12]
Zinc borate complex	6	72.5	------	.6	.01	------	15.6	------	72	11.9 boric oxide (B_2O_3).
Carbohydrates:										
Sucrose	1	66.7	------	[10] 96.3	.0	------	------	------	--------	1.4 phenol.

[1] Based on results shown in tables 1-4.

[2] Based on weight of sample as received.

[3] Approximate value obtained by the following empirical calculations: Methoxyl/0.128=lignin (TAPPI, T 629 m-53); lignin×1.154=lignosulfonic acid (TAPPI, T 629 m-53); lignosulfonic acid+cation equivalent of SO_3 in lignosulfonic acid=lignosulfonate salt.

[4] Calculated as glucose.

[5] Total carbohydrates—reducing sugars.

[6] May include such wood sugars as mannose, glucose, xylose, galactose, arabinose and fructose.

[7] Estimated by difference.

[8] Derived from Kraft process.

[9] Maximum possible. The empirical calculation gave unreasonably large values because of high methoxyl contents.

[10] Established as sucrose by infrared spectral analysis.

[11] Probably alkyl aryl sulfonate salt.

[12] Calculated from total nitrogen content.

Ultraviolet Spectral Analysis

Each material was analyzed by ultraviolet spectroscopy to determine if it could be identified or characterized by its ultraviolet absorption spectrum. The apparatus used for ultraviolet work included a double beam quartz spectrophotometer, an ultraviolet power supply unit, and a hydrogen discharge lamp as a light source. Measurements were made in 1 cm. matched silica rectangular cells.

Procedure for lignosulfonates

Special buffer.—495 ml. of 0.2 N KH_2PO_4 and 113 ml. of 0.1 N NaOH were mixed and diluted to 2 liters.

Sample preparation.—0.5 g. of solid sample or exactly 1 ml. of liquid sample was dissolved and diluted with water to 100 ml. Insoluble material was removed by centrifuging. A 10 ml. clear aliquot was diluted to 100 ml., and finally a 1 ml. aliquot of the latter was diluted to exactly 10 ml. with the buffer solution. Final concentration was 0.005 percent by weight or 0.01 percent by volume in the case of the liquid samples.

Measurements.—Absorbance for each material was measured at intervals between 220 and 350 millimicrons in 1 cm. cells. Readings

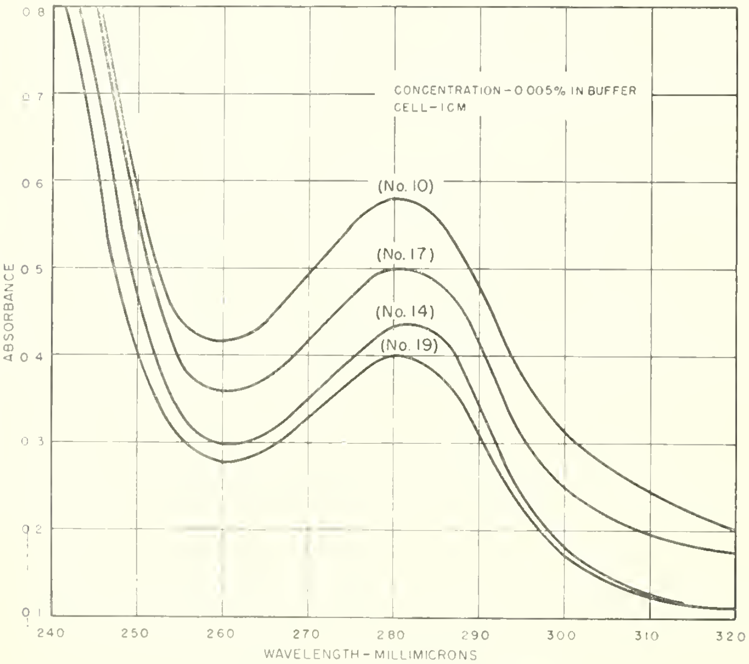

Figure 2.—Typical ultraviolet spectra of lignosulfonates.

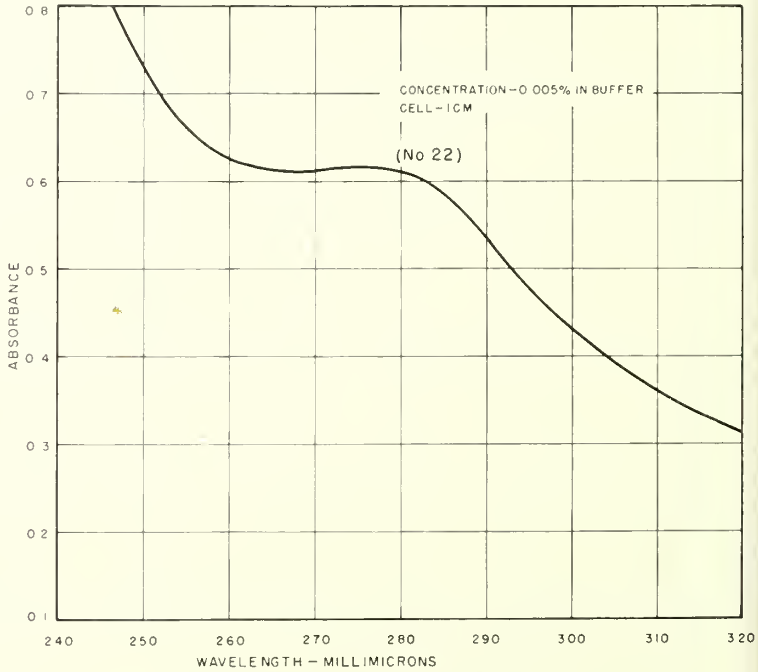

Figure 3.—Ultraviolet spectrum of lignosulfonate from Kraft process.

were made at wavelength intervals of 5 to 10 millimicrons, except where peaks appeared near 260 and 280 millimicrons. In these areas, readings were obtained at 0.5 to 1.0 millimicron intervals. The sensitivity of the instrument was adjusted so as to maintain the smallest slit openings. The lamp housing was cooled with circulating tap water.

Plotting.—The absorbance readings were plotted against wavelength and the resultant spectral curve was then drawn manually. Absorbance is defined as $\log_{10} I_o/I$, where I_o equals incident radiant power, and I equals transmitted radiant power.

Results of tests on lignosulfonates

Figure 2 shows examples of typical ultraviolet spectra obtained on several lignosulfonate retarders. The shape of each curve was typical of all the other lignosulfonate retarders except one, retarder No. 22, which had been derived from the Kraft process. The spectrum of sample No. 22, shown separately in figure 3, indicated a shoulder rather than a peak occurred at 280 millimicrons. It thus appeared that lignosulfonates as a class could be identified from their characteristic ultraviolet spectra. In addition, the height of the peak (absorbance) at 280 millimicrons could be utilized for quantitative information. Figure 4 shows that lignosulfonate concentration and absorbance values have a linear relationship in accord with Beer's law. These tests confirm previous reports that lignosulfonates may be analyzed quantitatively as well as qualitatively by ultraviolet spectrophotometry (*9*).

Table 6 gives the ultraviolet spectral data for all the lignosulfonate retarders tested. It can be seen from this table that the peaks for each material occurred within a narrow wavelength range.

Results on the other chemical types

None of the other types of retarders studied had a significant ultraviolet spectrum that was characteristic of the active constituent. Retarder No. 1 did have a characteristic spectrum which was produced by a minor constituent, phenol. Generally, however, ultraviolet spectral analysis was not found suitable for identifying organic hydroxycarboxylic acids or carbohydrates.

Investigation of Visible Spectra

The double beam quartz spectrophotometer with a tungsten lamp as a light source was used to investigate the visible spectra of the materials other than lignosulfonates. The spectral patterns obtained are shown in figure 5. Each spectrum is not sufficiently unique to differentiate between different retarder types or specific retarders. It was found that phenol, sucrose, and even triethanolamine gave spectra similar to those shown in figure 5 and, consequently, this approach was also not suitable for identifying retarders.

Infrared Spectra

Infrared spectral curves were obtained for each material by means of a double-beam spectrophotometer. The equipment used included a Perkin-Elmer double-beam Infrared Spectrophotometer, Model 137 (Infracord), with automatic recording and a sodium chloride prism for operation between 2.5 and 15 microns. Scanning time was approximately 12 minutes. An evacuable die was used to prepare samples by the potassium bromide disk technique.

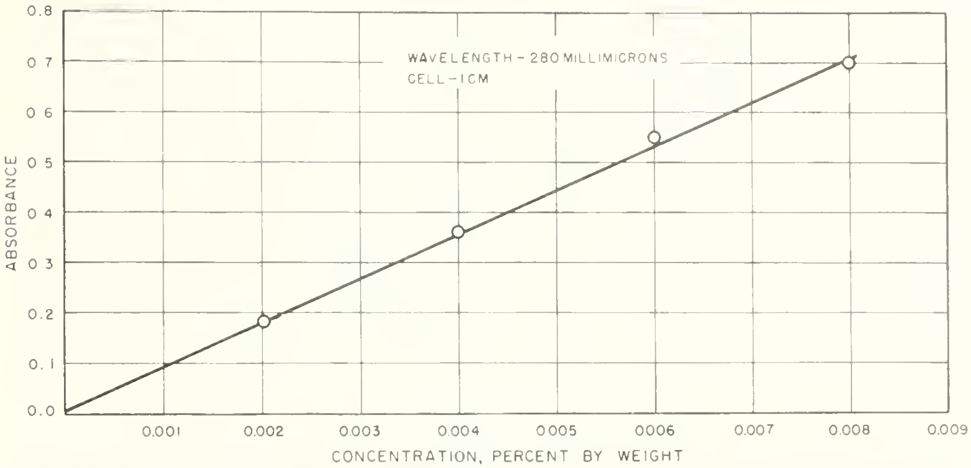

Figure 4.—Relation between concentration and absorbance for lignosulfonate retarder.

Procedure

The pressed disk technique was considered most suitable for the retarders studied because of the relative insolubility of these materials in appropriate organic solvents used in solution techniques. The mull method was discarded because of the effects of the mulling agent and the limited quantitative application of this method.

In the pressed disk method, solid samples were ground to a fine powder with mortar and pestle and then vacuum dried at 50° C. for at least 24 hours. Liquid samples were evaporated to dryness at a low temperature, ground, and dried under the same conditions. Approximately 1 mg. of sample and 0.35 g. of potassium bromide (anhydrous spectroscopic grade, 200/325 mesh) were weighed into a special stainless steel capsule. Two stainless steel balls were added and the contents mixed for 30 seconds by an electric amalgamator.

The powder was transferred to the evacuable die (shown disassembled in figure 6), and the assembly was evacuated to an absolute pressure of less than 1 cm. of mercury. Vacuum was maintained for 5 minutes prior to pressing as well as during pressing. A 1,000-pound load was applied for 1–2 minutes followed by a 20,000 pound load for 3–5 minutes. The potassium bromide disk was then removed and analyzed in the infrared spectrophotometer. The disk measured 13 millimeters in diameter and was approximately one millimeter thick and is shown in figure 7.

A few of the dried retarder samples were tacky or viscous. These were slurried with alcohol, mixed with potassium bromide, vacuum dried, and then reground and pressed into disks.

Table 6.—Ultraviolet spectral data for lignosulfonate retarders

Retarder No.	Final concentration[1] (percent)	Spectral peaks[2]				Relative concentration of lignosulfonate, percent by weight[7]	
		Maximum		Minimum			
		Wavelength (millimicrons)	Absorbance	Wavelength (millimicrons)	Absorbance	Based on original material	Based on dry solids
2	0.05	281.0	0 151	260.5	0.121	2.0	2.0
3	.005	278.0	.457	262.5	.396	60.9	65.3
7	.005	280.5	.589	259.5	.431	78.5	82.2
10	.005	280.0	.582	260.0	.419	77.6	81.8
11	.005	283.0	.362	260.0	.225	48.3	51.5
12	.005	280.0	.474	261.0	.395	63.2	66.1
13	.005	279.0	.328	262.0	.292	43.7	47.9
14	.005	282.0	.438	261.0	.298	58.4	62.4
15	.005	278.0	.442	262.0	.390	58.9	63.6
16	.005	281.5	.408	260.5	.281	54.4	57.4
17	.005	280.5	.502	260.0	.359	66.9	72.4
18	.005	281.0	.393	260.5	.272	52.4	56.5
19	.005	280.0	.400	260.0	.278	53.3	56.3
20	[5] .01	279.0	.333	262.5	.284	19.1	61.6
21	[5] .01	280.0	.290	260.5	.248	16.8	56.6
22[3]	.005	[6] 276.0	.618	[6] 267.5	.611	82.4	90.1
23	.005	280.0	.460	260.5	.381	61.3	66.1
24	.005	284.0	.381	262.5	.297	50.8	52.9
25	.005	281.5	.441	261.0	.308	58.8	62.9
Median[4]	--------	280.0	----------	260.5	----------	-------	--------

[1] In 0.2N KH_2PO_4–0.1N NaOH buffer solution.

[2] 1 cm. cell.

[3] Derived from the Kraft process.

[4] Retarder No. 22 not included.

[5] Percentage by volume (original material was liquid).

[6] No sharp maximum and minimum, but rather a shoulder.

[7] Calculated from absorbance at maximum peak. Relative to retarder No. 2 which was assumed to be 2 percent.

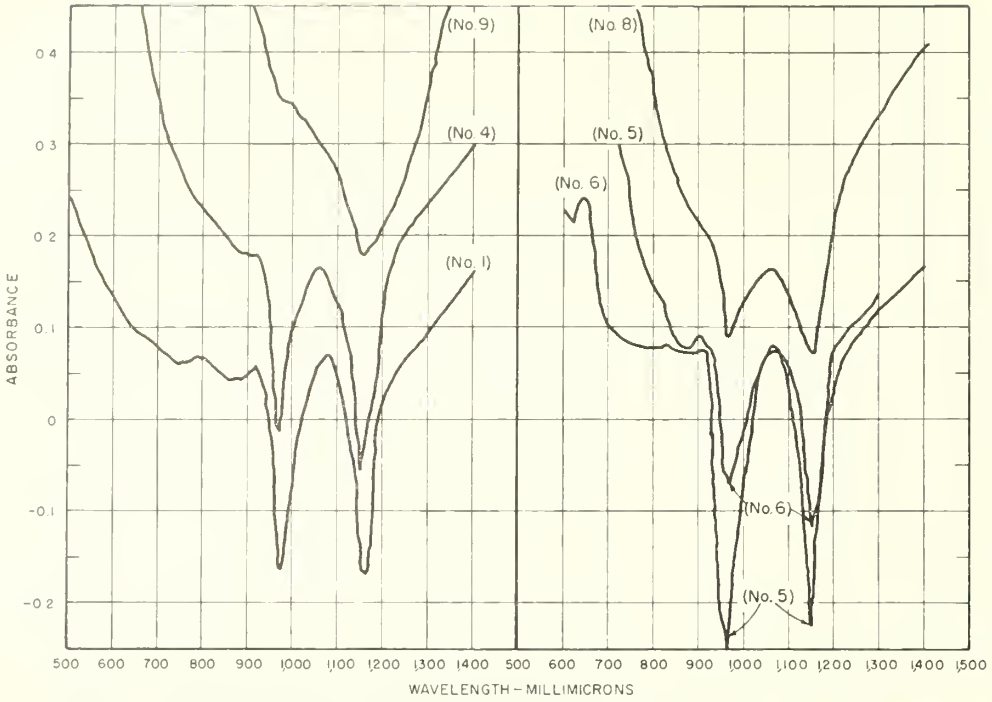

Figure 5.—Visible spectra of retarders containing organic acids or carbohydrates.

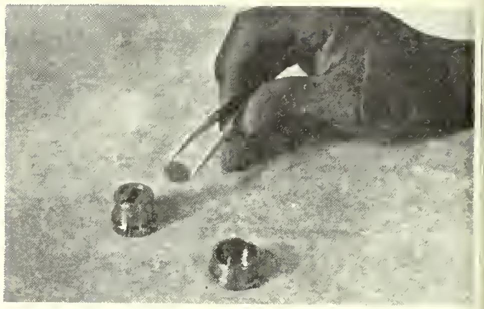

Figure 7.—Potassium bromide disk and disk holder.

Results of infrared analyses

Figures 8 through 12 illustrate typical infrared spectra of the different retarders. Each retarder gave a characteristic spectrum which could be used both to identify the material and establish the concentration of major active constituents. In general, lignosulfonate retarders had the same characteristic spectra regardless of the type of salt (i.e., sodium, calcium, or ammonium). Nevertheless, certain lignosulfonates containing carbonates or other major modifiers, as well as a lignosulfonate derived from the Kraft process, could be easily distinguished by their unique spectra.

Figure 8 shows the type of spectra obtained for the lignosulfonate type retarders. The top spectrum (A), retarder No. 12, illustrates the typical spectrum of lignosulfonates derived from the sulfite liquor or acid process. The characteristic peaks at different wavelengths in the lignosulfonate spectrum are produced by the following chemical groupings present:

1. Intense peaks at 2.9 and 9.6 microns: hydroxyl (OH) groups.
2. Moderate peak at 3.4 microns: usual carbon-hydrogen stretching bonds.
3. Strong peaks at 6.25 and 6.62 microns: carbon-carbon bonds (phenyl ring).
4. Weak peaks at 6.9 and 7.3 microns: probably sulfur-oxygen bond (sulfone group).
5. Broad band at approximately 8.3 microns: sulfonate group.

Spectrum B, figure 8, is the curve for lignosulfonate retarder No. 13. The presence of substantial amounts of sodium carbonate produced strong bands which masked part of the characteristic lignosulfonate spectra. The broad peaks at 7.0 and 11.3 are characteristic of the sodium carbonate present (see figure 9A which illustrates the infrared pattern of sodium carbonate). If desired, the sodium carbonate interference may be removed by neutralization with hydrochloric acid, followed by an alcohol extraction of the sodium chloride thus formed. The lignosulfonate is insoluble in alcohol and should then give a good characteristic spectrum.

Curve C in figure 8 represents a lignosulfonate (No. 22) obtained from the Kraft process for making paper. Although the curve shows the major peaks of a typical lignosulfonate, several additional characteristics help identify this material. For instance, at 8.3 microns (sulfonate group) absorption was at a greater intensity, and at 8.8 and 10.2 microns peaks were produced, probably by inorganic sulfate. Another characterization of this material was the weak absorption peak at 12.7 microns.

The last two spectra, D and E, in figure 8 are of the same sample (No. 2), which contained a lignosulfonate. Curve D was obtained on the original sample, and it is

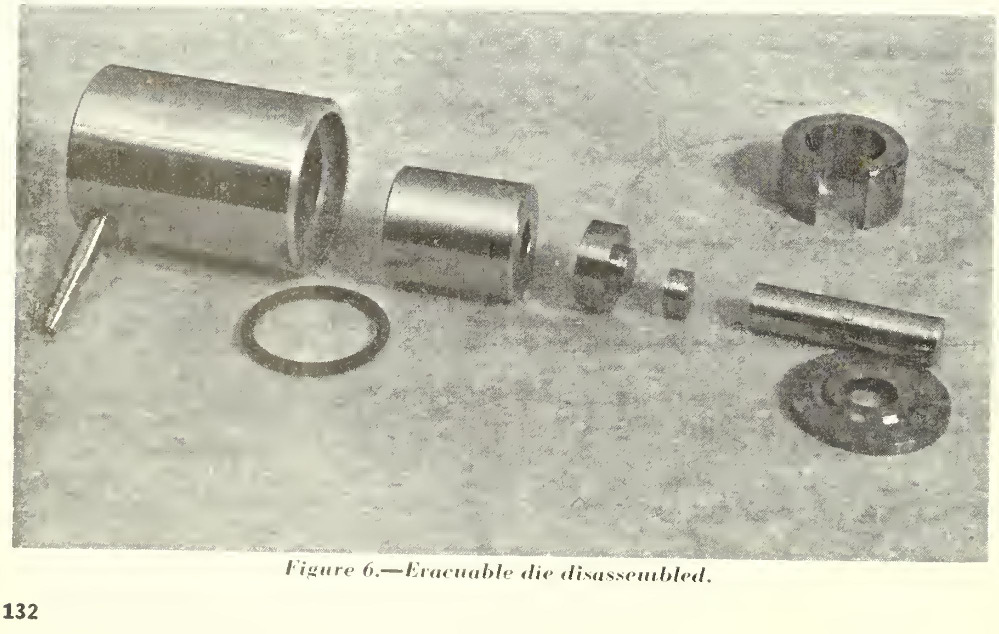

Figure 6.—Evacuable die disassembled.

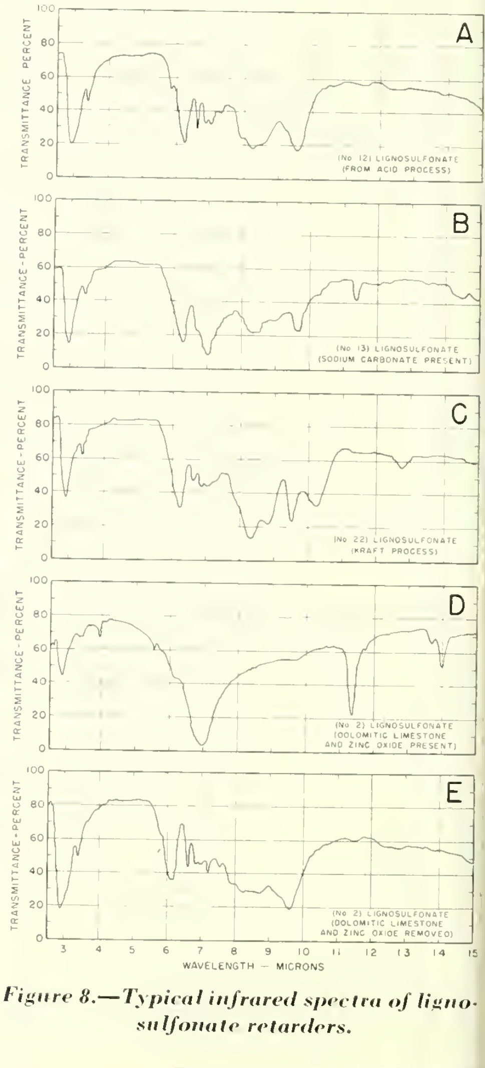

Figure 8.—Typical infrared spectra of lignosulfonate retarders.

apparent that it does not clearly show the characteristic lignosulfonate pattern. This was caused by the presence of a large quantity of dolomitic limestone which produced an intense spectrum of its own. When this constituent was removed by centrifuging an aqueous suspension, the characteristic spectrum of lignosulfonate was evident as shown in curve E. Spectrum D is useful in that it supplied ample evidence of the presence of dolomitic limestone in the original material. The peak at 14 microns was unique for calcium carbonate (limestone), while the smaller peak at 13.7 microns was unique for dolomite. From the relative intensities of these two peaks it was estimated that the ratio of dolomite to limestone was approximately 1:4.

Figure 9 shows the spectra obtained on several carbonate materials, namely, sodium carbonate, calcium carbonate, and dolomite. Spectra B and C illustrate the characteristic peaks for limestone and dolomite discussed above.

Figure 10 illustrates the unique infrared patterns which may be used to identify different hydroxy-carboxylic acid salts. It is quite apparent that these curves are distinctly different from the lignosulfonate pattern as well as from each other. Spectrum A (retarder No. 8) shows the masking effect of a large amount of siliceous matter and iron oxide present in the material. To eliminate this interference, an aqueous suspension of the retarder was centrifuged to remove insoluble siliceous material and iron oxide. The remaining material then gave a distinctive infrared pattern of organic material as seen in spectrum B. The following chemical groups accounted for the more significant peaks in spectrum B: (1) Hydroxyl—intense peak at 3.1 microns; (2) carbon-hydrogen stretching—minor peak at 3.4 microns; (3) carboxyl and carboxyl salt—intense peaks at 6.3, 9.1, and 9.6 microns; and (4) overtones of the carbon-hydrogen linkages accounted for the other peaks from 7.3 to 8.3 microns.

The spectrum of a triethanolamine salt of a hydroxy-carboxylic acid is shown in spectrum C, figure 10. Although the major peaks of hydroxyl, carbon-hydrogen, and carboxylic groups were evident, this spectrum had sufficiently unique features to clearly identify the retarder. For instance, there was a prominent peak at 10.9 microns, probably caused by a carbon-nitrogen bond, and the usual hydroxyl peak at 3.0 microns was accentuated by the presence of nitrogen-hydrogen groups in this material.

Figure 11 shows the spectra of still another organic acid retarder (No. 6). Curve A shows the spectrum of the original material, whereas Curve B was obtained after zinc and borates were removed. Zinc was removed by two batchwise treatments with a cation exchange resin, 200–400 mesh hydrogen-form (Amberlite IR–120 or Dowex–50) followed by vacuum distillation with methyl alcohol to remove the boron as volatile methyl borate (*10*). Spectrum B shows many of the usual peaks of a hydroxy-carboxylic acid, and is sufficiently distinctive to be used to identify this material. The prominent peak at 5.6 microns was undoubtedly caused by a lactone formation. Spectrum C in the same figure is that of a technical grade of gluconic acid which showed a striking resemblance to sample No. 6 (spectrum B).

The infrared spectrum of the carbohydrate retarder is illustrated in figure 12. Spectrum A (retarder No. 1) presents a pattern that was quite unique and therefore useful for identification. A comparison of this spectrum with the spectrum of sucrose or cane sugar (curve B) clearly demonstrates that retarder No. 1 is essentially sucrose.

Although no effort was made in this report to use the infrared spectra for quantitative analysis of the materials, such techniques could be easily applied. For solid samples, such as potassium bromide pellets, the baseline technique is most appropriate and has been well described. Generally, this technique involves the measurement of the depth of a single significant peak, compared to a reference base line.

Uniformity of trade products

The ability of infrared analysis to "fingerprint" or measure the uniformity of different batches of specific proprietory products is illustrated by figures 13 and 14. Figure 13 shows the spectra of four different lots of a solid lignosulfonate retarder sold under one trade name. Each material had been obtained from a supplier by different State highway departments at different times ranging from 1954 (fig. 13A) to 1958 (fig. 13D). The general shape of the spectra are the same, with signifi-

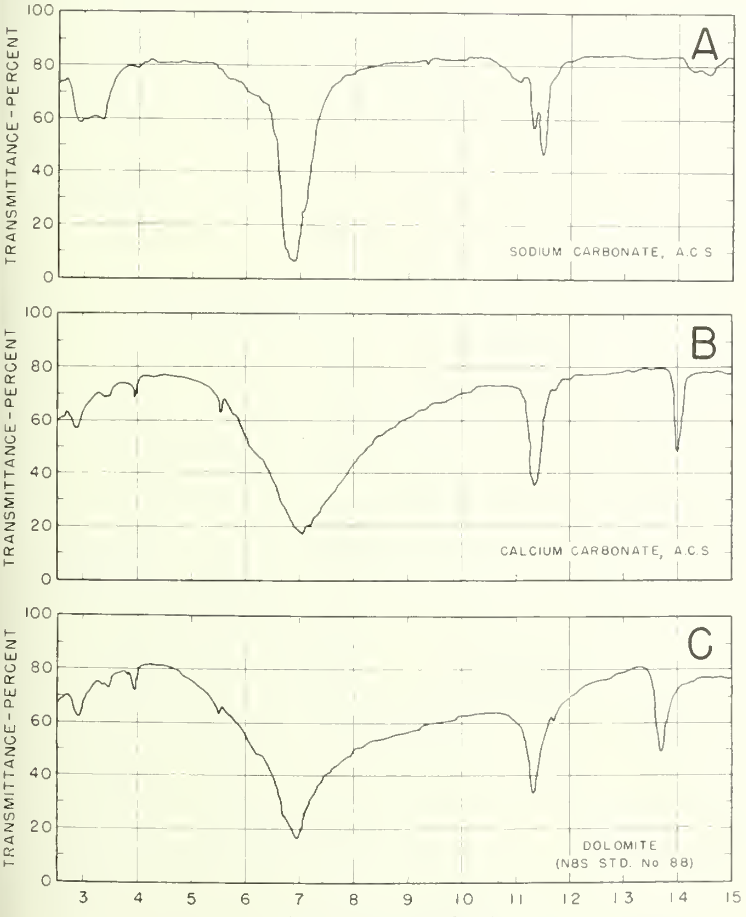

Figure 9.—Infrared spectra of related inorganic carbonates.

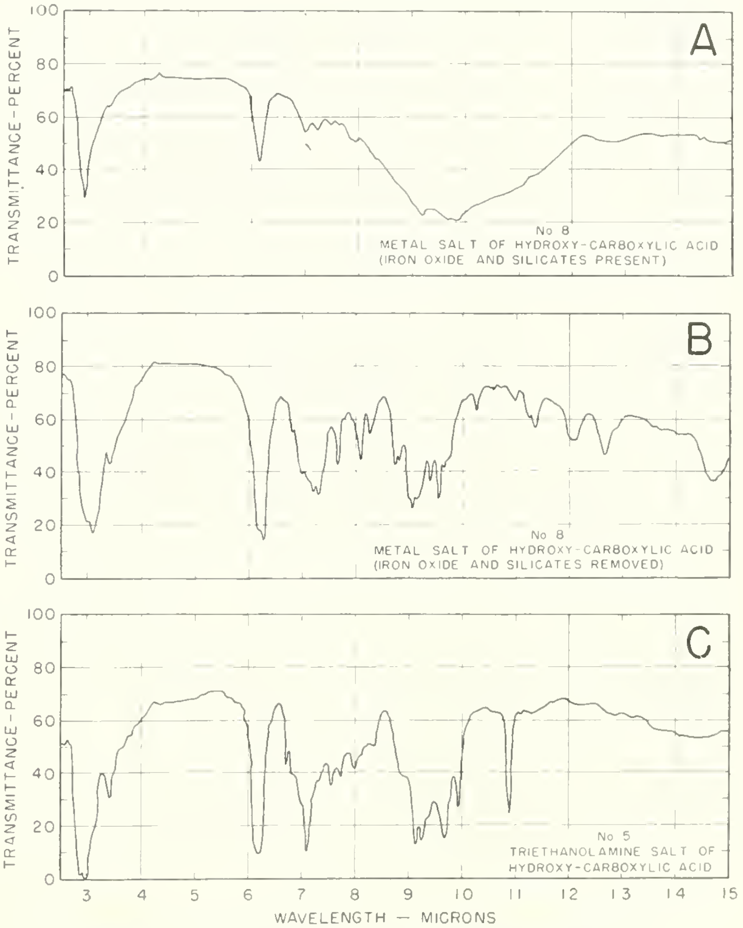

Figure 10.—Infrared spectra of organic acid retarders.

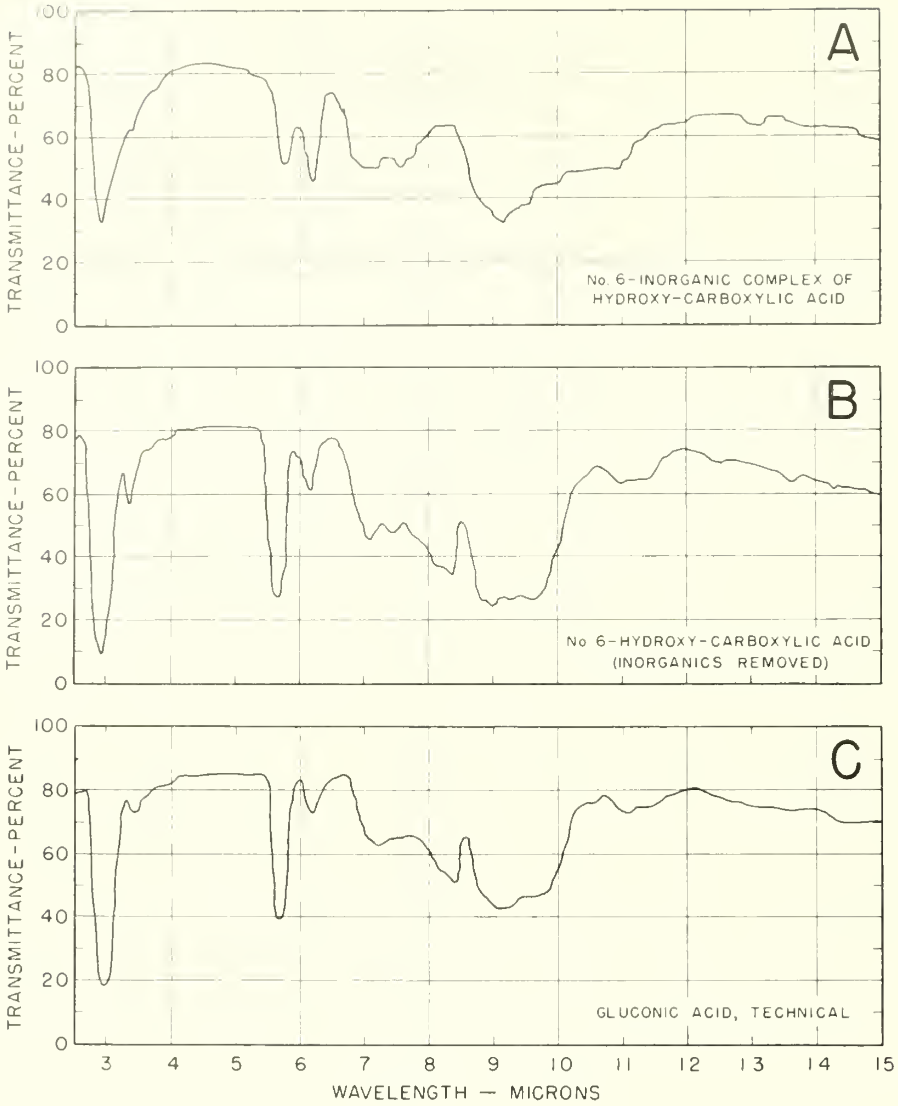

Figure 11 (to the left).—Infrared spectra of complexed organic acid retarder and gluconic acid.

cant peaks occurring at the same wavelengths in each lot. This definitely established that in each case the materials were chemically the same. By analyzing the peak intensities at selected wavelengths and knowing the concentration of sample used in the infrared analysis, any material differences in composition of the retarder from batch-to-batch could be demonstrated. Here, the compositions were shown to be fairly uniform, thus establishing that no material alteration or differences existed between the lots submitted.

Figure 14 shows the spectra for an organic acid retarder, specifically, a hydroxy-carboxylic acid salt in liquid form. Here again, each sample was obtained under the same trade name by different State highway departments at different times ranging from 1956 (fig. 14A) to 1958 (fig. 14D). The "fingerprinting" ability of infrared analysis once again determined the nature and concentration of the ingredients. The uniformity of the spectra shows that each lot was substantially the same.

Comparison of Methods of Analyses

Since the major objective of this study was to develop procedures by which the composition of commercial retarders could be readily identified and determined so as to provide a basis for obtaining the necessary assurance that the composition would be uniform from batch-to-batch, it is noteworthy to compare the various methods of analyses.

Infrared spectral analyses offers the most promising and rapid means of clearly identifying and classifying retarder materials. This technique, by obtaining recorded spectral curves, "fingerprints" the unique and distinctive characteristics for each retarder. Thus, carbohydrates and organic hydroxy-carboxylic acids were easily identified due to their distinctive spectral patterns. Lignosulfonates likewise had unique characteristics which made identification easy, regardless of the type or salt present or the source of supply. Further, spectral difference caused by the manufacturing process or the presence of other ingredients aided in the identification and determination.

Infrared analyses can also be used to assure the purchaser that the nature and concentration of each lot of retarder for specific field projects has not been materially altered from that of the original material. The time required for the analysis is only 20 to 30 minutes as compared to a week or more by conventional methods of chemical analysis.

Ultraviolet techniques were also found to be of value in identifying lignosulfonates and in establishing the concentration of the major active ingredient. However, specific commercial lignosulfonates were not as easily differentiated by ultraviolet spectra as compared to their infrared spectra. Ultraviolet as well as visible spectral analyses were not found suitable for identifying other types of retarders.

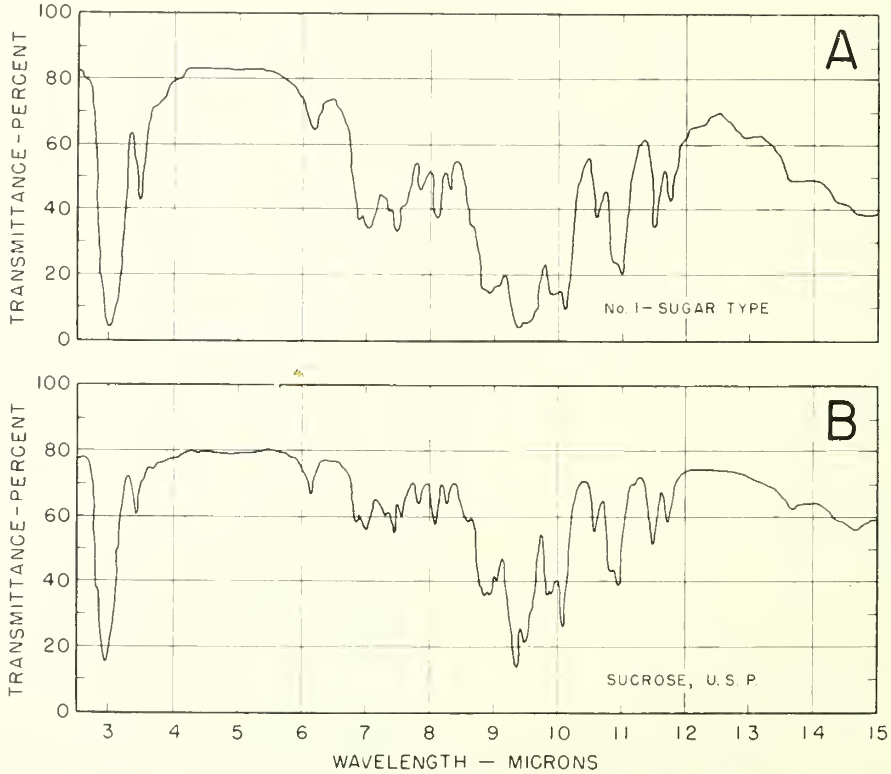

Figure 12.—Infrared spectra of carbohydrate retarder and sucrose.

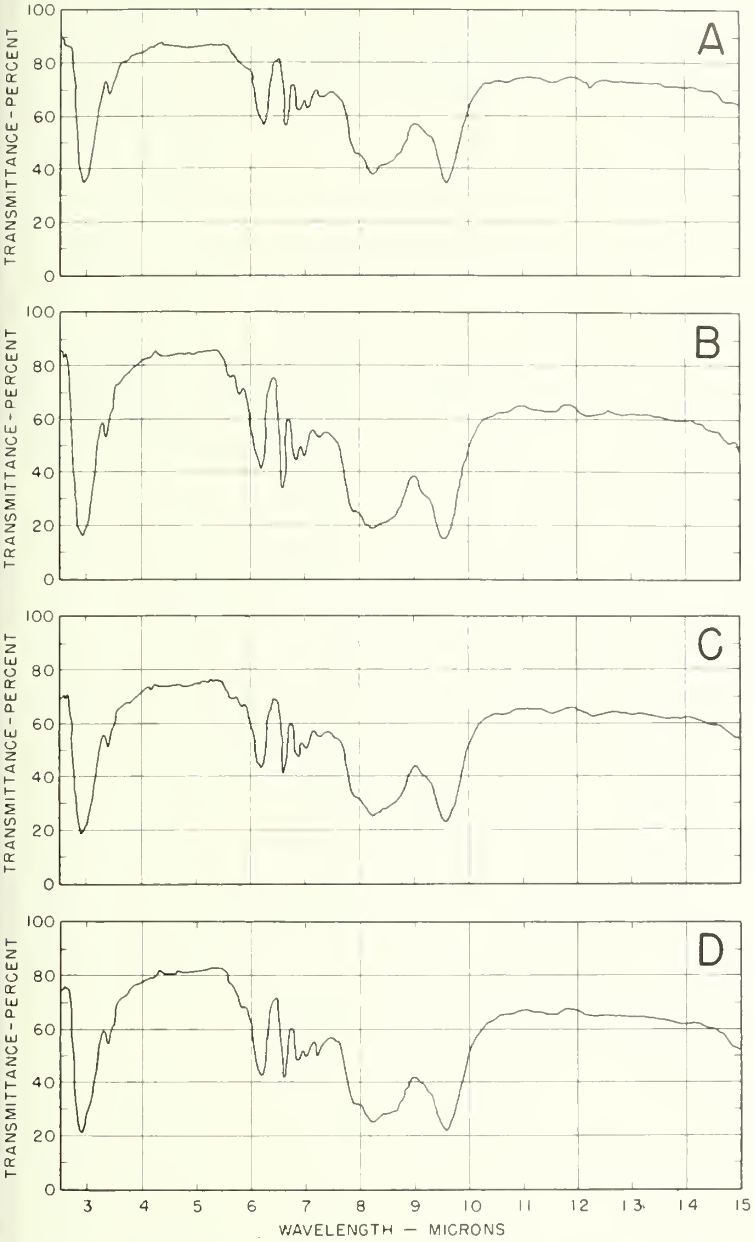

Figure 13.—Infrared spectra of four different lots of a lignosulfonate retarder obtained from one manufacturer.

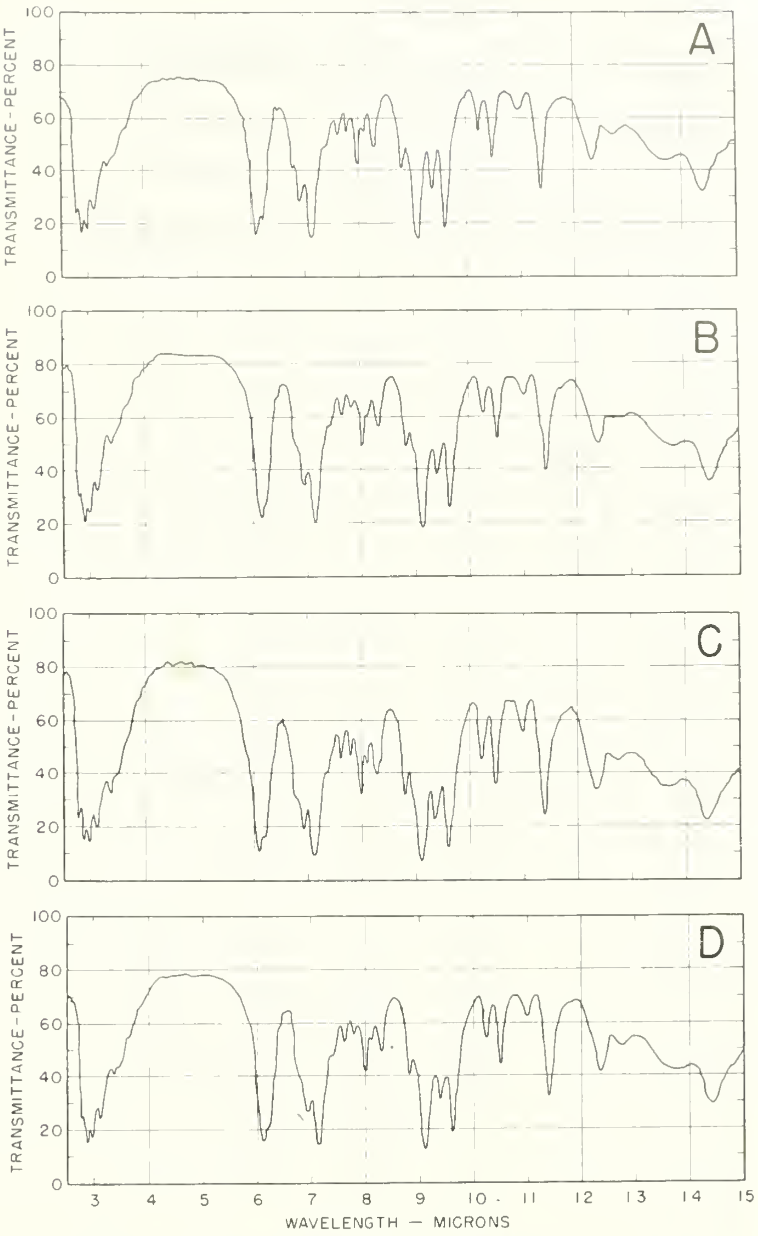

Figure 14.—Infrared spectra of four different lots of an organic acid retarder obtained from one manufacturer.

The determination of the quantitative amounts of inorganic constituents can be most conveniently and precisely determined by conventional chemical methods. However, while useful, the conventional procedures were tedious and time-consuming, and often yielded empirical or doubtful results for certain organic constituents. This was found to be particularly true among the lignosulfonate and organic acid retarders.

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Water-Reducing Retarders for Concrete—Physical Tests

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported[1] by
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DONALD O. WOOLF,
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THE USE OF CHEMICALS for delaying the initial setting time of portland cement concrete is not new. The effects of sugar and other retarders on concrete have been known for many years. In 1945, it was reported that a retardant admixture had been used during 1942 to 1945 in lightweight concrete for the construction of concrete ships (*1*)[2]. The admixture was used in non-air entrained concrete to reduce the amount of mixing water, or to increase slump and to prevent the early stiffening and formation of "cold joints."

Since that time there have been many articles written discussing the use of retardants in concrete for prestressed beams or piles and bridge structures and relating valuable test data (*2*, *3*).

Several Federal agencies have been using temporary or interim specifications for retarders. The Bureau of Reclamation suggested a specification in 1954 and the Bureau of Public Roads proposed a specification for certain local usage in 1956 which was revised in 1958. These specifications were based mainly on field experiences with retarders from two manufacturers, and on a limited amount of laboratory test data. The need for a specification for commercially available retarders has become greater since the possible benefits of retarders have been recognized more fully.

Concrete Materials Used

The classification of the 25 retarders tested and their general properties are shown in table 1. The information supplied by the marketer and tests by the Bureau were the bases for the classification.

A blend of equal parts by weight of four brands of type I cement was used for all concretes in this investigation. The chemical composition and physical properties of each brand and of the blend are shown in table 2. About 32 bags (one lot) of each of the four cements were stored in sealed 55-gallon steel drums until used, and three lots of each of the four cements were used altogether during the investigation.

The air entraining admixture used was an aqueous solution of neutralized Vinsol resin. The solution contained 15 percent of solids when calculated as a residue dried at 105° C., and the ratio of sodium hydroxide to Vinsol resin was 1 to 6.4 parts by weight.

A natural siliceous sand having a fineness modulus of 2.90, a specific gravity (bulk dry) of 2.65, and an absorption of 0.4 percent, and a crushed limestone of 1-inch maximum size, having a specific gravity (bulk dry) of 2.75 and an absorption of 0.4 percent, were used as aggregates. When the aggregates for a batch of concrete were weighed, the sand contained free water but the stone was weighed in a saturated surface-dry condition.

Three mixes of air-entrained concrete with air contents of 5 to 6 percent and a slump of 2 to 3 inches were used in the study. Mix No. 1, the reference mix without retarder, and mix No. 2, a test mix with retarder, had cement contents of 6 bags per cubic yard of concrete. Mix No. 3, also a test mix with retarder, had cement contents which varied from 5.25 to 5.75 bags per cubic yard and the water-cement ratio was approximately the same as that of the reference mix (No. 1). The water content of mix No. 2 was reduced below that of the reference mix but sufficient aggregates were added to compensate for the reduced volume. A summary of the mix proportions is given in table 3.

A sufficient amount of retarder was used in the tests of mix No. 2 at 73° F. to cause a delay in setting time of 2½ to 3 hours beyond the setting time of the reference mix. The correct amount of each retarder for the desired retardation was predetermined by trial mixes. The same amount of retarder, in ounces per

Table 1.—Properties of retarders

Type of retarder	Retarder No.	Physical state	Amount of retarder: Recommended by manufacturer	Amount of retarder: Used [1]	Retarder constituents [2]: Dry solids [3]	Retarder constituents [2]: Volatile organic material [4]
Lignosulfonates:			*Oz./bag*	*Oz./bag*	*Oz./bag*	*Oz./bag*
Ammonium salts	7	Powder	------	3.5	3.3	3.3
	10	do	3.8	3.2	3.0	3.0
	[5,6] 17	do	------	3.2	3.0	2.9
Sodium salts	15	Powder	3.8- 7.5	4.0	3.7	2.8
	19	do	4.0	4.0	3.8	2.8
	[7] 22	do	.8	4.0	3.7	2.0
	23	do	3.8	6.4	5.9	3.9
Calcium salts	[8] 2	Powder	8.0	14.4	14.3	.40
	3	do	3.8- 7.5	4.0	3.7	3.1
	[9] 11	do	4.0	4.8	4.5	3.2
	12	do	--------	4.0	3.8	3.0
	13	do	4.5- 7.5	4.8	4.4	2.3
	[6] 14	do	3 - 4	5.0	4.7	4.0
	16	do	4.0	4.8	4.5	3.8
	18	do	4.0- 5.0	4.0	3.7	3.4
	20	Liquid	9.0	9.0	3.3	2.6
	21	do	9.0	9.0	3.2	2.5
	[9] 24	Powder	12.0	9.6	9.2	5.1
	[5,10] 25	do	3 - 4	4.0	3.7	3.1
Organic acids:						
Metal salts	4	Liquid	2.0- 4.0	3.0	1.2	.88
	8	Powder	8.0-16.0	12.0	11.9	1.2
	[11] 9	Liquid	2.0- 4.0	2.8	1.1	.77
Triethanolamine salt	5	Liquid	2.7- 7.1	4.0	2.1	2.1
Zinc borate complex	6	do	2.0	4.0	1.3	.91
Carbohydrates: Sucrose	[5] 1	Liquid	1.0- 4.0	1.1	.4	.43

[1] Amount of retarder necessary to retard set of concrete from 2½ to 3 hours as determined by Proctor penetration test at 500 p.s.i.
[2] Additional chemical constituents reported in footnotes.
[3] Determined by drying at 110° C.
[4] Determined by ignition.
[5] Over 10 percent carbohydrate.
[6] Over 5 percent reducing sugar.
[7] Derived from Kraft process.
[8] Contained less than 5 percent calcium lignosulfonate.
[9] Contained calcium chloride.
[10] Contained foaming agent.
[11] Contained less than 10 percent of active organic constituent. Remainder is iron oxide and siliceous material.

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[1] Presented at the 40th Annual Meeting of the Highway Research Board, Washington, D.C., January 1961.

[2] Italic numbers in parentheses refer to a list of references on page 154.

Table 2.—Chemical composition and physical properties of portland cement

Determination [1]	Cement A	Cement B	Cement C	Cement D	Blend [2]
Chemical composition (percent):					
Silicon dioxide	21.4	21.4	21.9	21.4	21.6
Aluminum oxide	5.9	5.1	5.6	5.4	5.4
Ferric oxide	2.8	2.2	2.3	2.4	2.4
Calcium oxide	63.4	62.5	66.1	65.2	64.3
Magnesium oxide	2.1	3.8	1.2	1.6	2.2
Sulfur trioxide	1.8	2.4	1.7	1.9	2.0
Loss on ignition	1.7	1.5	1.2	1.1	1.4
Sodium oxide	.20	.30	.02	.14	.18
Potassium oxide	.82	.95	.14	.79	.69
Equivalent alkalies as Na_2O	.74	.93	.11	.66	.63
Chloroform-soluble organic substances	.016	.008	.004	.007	.008
Free lime	.97	.93	.97	.66	.94
Computed compound composition (percent):					
Tricalcium silicate	47	48	57	58	52
Dicalcium silicate	26	26	20	18	23
Tricalcium aluminate	11	10	11	10	10
Tetracalcium aluminoferrite	8.5	6.7	7.0	7.3	7.4
Calcium sulfate	3.05	4.08	2.89	3.23	3.31
Merriman sugar test:					
Neutral point, ml	7.6	2.4	2.3	2.5	3.3
Clear point, ml	8.8	2.8	2.6	2.8	4.2
Physical properties:					
Apparent specific gravity	3.12	3.13	3.14	3.14	3.12
Specific surface (permeability), cm.^{2}/g	3,375	3,590	3,110	3,145	3,310
Autoclave expansion, percent	.07	.19	.04	.06	.07
Normal consistency, percent	25.3	24.3	24.4	24.6	24.0
Time of setting (Gillmore test):					
Initial, hours	2.9	2.8	3.4	4.0	2.3
Final, hours	6.2	5.1	6.8	6.0	7.5
Compressive strength:					
At 3 days, p.s.i	2,500	2,500	2,030	2,610	2,490
At 7 days, p.s.i	4,165	3,450	3,225	4,175	3,840
At 28 days, p.s.i	5,690	4,565	5,585	5,510	5,285
Tensile strength:					
At 3 days, p.s.i	340	320	300	320	315
At 7 days, p.s.i	400	395	365	435	400
At 28 days, p.s.i	470	465	475	465	480
Mortar air content, percent	8.0	8.8	7.5	7.8	8.6

[1] All determinations except the Merriman sugar test were made in accordance with current AASHO methods for portland cement.
[2] Tests made on blend of equal parts by weight of all four cements.

bag of cement, was used in mix No. 3. Additional tests of some retarders were made using four times the amount of the retarder used in mixes Nos. 2 and 3.

Mixing, Fabrication, and Curing of Specimens

Mixing was done in an open-pan type Lancaster mixer with a rated capacity of 1¾ cubic feet. Most batches were from 1½ to 2 cubic feet in volume. The mixing cycle was as follows: The blend of four cements and the moist fine aggregate were mixed for 30 seconds; water was added and the mortar was mixed for 1 minute. After the addition of the coarse aggregate the concrete was mixed for 2 minutes. Following a rest period of 2 minutes, the concrete was mixed for 1 more minute. This 2-minute rest period and the additional minute of mixing is standard laboratory procedure for the Bureau of Public Roads.

Retarder in powder form was added with the cement and sand, but a liquid retardant admixture was added with part of the mixing water. Vinsol resin solution was added with part of the mixing water, but was not mixed with the liquid retarder until each was placed in the mixer.

Consideration was given to adding the soluble powdered admixtures in an aqueous solution. Although the soluble powders could have been prepared in a stock solution or suspension, it was believed that during the period of months required for the program, evaporation or chemical changes of the solution of the retarder due to exposure to light might result. An attempt was made to dissolve in water the weighed amount of lignosulfonate powder required for a single batch of concrete but difficulty was experienced with some powders in obtaining a uniform solution or suspension. Therefore, the procedure of adding the powdered admixture with the cement and wet sand was used. It was believed that a uniform distribution of the powdered admixture in the concrete was obtained.

A control or reference mix (No. 1) was made on each mixing day together with the retarder mixes (Nos. 2 and 3). Due to time and mold limitations only one control mix and two retarder test mixes were made on each mixing day for each of three to five different retarders. Two rounds of specimens for retardation tests and five rounds for strength tests were made but each round did not necessarily include the same retarders as in the preceding round.

Concrete used for slump and unit weight tests was returned to the mixer and remixed for 15 seconds before molding specimens for strength tests. Concrete used in the air meter was discarded. Specimens made at standard conditions were mixed and molded in a temperature of 73° F. and 50 percent relative humidity.

Specimens for strength or durability tests were 6- by 12-inch cylinders and 6- by 6- by 21-inch or 3- by 4- by 16-inch beams. These were molded (except for vibrated specimens) in accordance with standard AASHO methods. Specimens for strength tests were removed from the steel molds at 20 to 24 hours. Small beams for volume change and freezing and thawing were removed from the steel molds at 44 to 48 hours.

For a study of delayed vibration, 6- by 12-inch cylinders and 6- by 6- by 21-inch beams were molded by the standard procedure and later vibrated for 30 seconds using an internal spud vibrator of 1⅛-inch diameter with a frequency of 7,000 impulses per minute.

The original program required that the specimens be vibrated when the screened mortar showed a Proctor penetration[3] load of 500 p.s.i., which was approximately 5 hours after mixing for unretarded concrete and 7½ to 8 hours for the retarded concretes. However, the prescribed time intervals proved too long as the mixes became too stiff for vibration. Therefore, the time intervals were reduced to 3 hours after mixing for the nonretarded concrete and 5½ hours for the retarded concrete. A small amount of concrete was removed from each specimen just prior to vibration and it was replaced as the vibration was completed. No additional concrete was added to compensate for the decrease in volume.

All specimens for standard strength tests were cured under wet burlap while in the molds in the mixing room. After removal from the molds they were cured in moist air at 73° F. and 100 percent relative humidity until they were tested. Small beams for volume change and freezing and thawing were cured in the molds for 44 to 48 hours under wet burlap in the moist room. One-third of the beams for volume change was cured continuously in the moist room, one-third were removed from the moist room at 2 days and stored in laboratory air at 73° F. and 50 percent relative humidity, and one-third were cured 14 days in the moist room and

Table 3.—Summary of concrete mix proportions

Properties of concrete	Mix No. 1	Mix No. 2 [1]	Mix No. 3 [2]		
			Group A	Group B	Group C
Mix:					
Cement, pounds	94	94	94	94	94
Sand, pounds	190	193	205	215	230
Limestone, pounds	310	314	330	345	360
Cement, bags per cubic yard	6.0	6.0	5.75	5.50	5.25
Water, gallons per bag	5.8	5.1–5.7	5.8	5.8	5.8
Water-cement ratio, by volume	.77	.68–.76	.77	.77	.77
Slump, inches	2½	2½	2½	2½	2½
Air, percent	5.5	5.5	5.5	5.5	5.5
Retarder, amount used	None	(3)	(3)	(3)	(3)

[1] Mix No. 2 had same cement content as Mix No. 1.
[2] Mix No. 3 had same water-cement ratio (±0.2 gal/bag) as Mix No. 1. Group A—cement content reduced 0.25 bag per cubic yard for retarders that reduced water for Mix No. 2 by up to 5 percent. Group B—cement content reduced 0.50 bag per cubic yard for retarders that reduced water for Mix No. 2 by 5 to 10 percent. Group C—cement content reduced 0.75 bag per cubic yard for retarders that reduced water for Mix No. 2 by 10 percent or more.
[3] To give 2½ to 3 hours retardation.

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[3] A method of testing for rate of hardening of mortars sieved from concrete mixtures by Proctor penetration resistance needles in accordance with ASTM Method C 403–57 T. This method is referred to as the Proctor penetration method.

then stored in laboratory air. Beams for freezing and thawing, after removal from the molds at 44 to 48 hours, were cured in the moist room for 12 more days, then cured in laboratory air at 73° F. and 50 percent relative humidity for 7 days and then were completely immersed in water for 7 days prior to freezing.

When strength specimens were made under nonstandard atmospheric conditions, the laboratory air temperature was 90° F. and the relative humidity was 20 to 25 percent. All materials were at 90° F. when used. After 20 to 24 hours curing under wet burlap at this temperature, the specimens were removed from the molds and stored in the moist room at 73° F. and 100 percent relative humidity until tested.

Testing Procedures

Tests on the plastic concrete for slump, unit weight, and air content were made in accordance with AASHO standard methods. The determination of the air content was made with a water-type pressure meter that is similar to the meter described in AASHO Method T 152.

Determination of the retardation of the set of concrete was made by the Proctor penetration method and by the bond pin pullout method.[4] The Proctor penetration apparatus with a hydraulic indicating dial and the pin pullout device are shown in figures 1 and 2. Both apparatus were constructed by the Bureau of Public Roads.

The mortar for the Proctor penetration method was obtained by sieving the plastic concrete on a No. 4 sieve. The sieve was vibrated and the concrete was moved over the sieve by hand or by use of a small trowel. Two 6- by 6-inch watertight steel cylinder molds were filled with mortar, and were immediately covered with glass plates. Water was pipetted from the mortar as it collected on the surface.

Concrete for the pin pullout test was vibrated around the pins in a 6- by 6- by 24-inch mold using a laboratory internal vibrator, and was screeded on the surface by a steel straight edge. The specimens were covered with wet burlap which was removed only while a pin was pulled from the concrete. No two consecutively pulled pins were adjacent to each other.

Typical curves for retardation of mixes Nos. 1 and 2 by the Proctor penetration and pin pullout methods are shown in figure 3.

Two series of tests were made to determine the temperature rise of concrete prepared with retarders. In the first series of tests, 6- by 6-inch cardboard cylinder molds were filled with screened mortar from concrete that was mixed at 90° F. The specimens were then sealed by a glass plate and heavy grease and stored in a curing cabinet at 90° F. In the second series, 6- by 12-inch cardboard cylinder molds were filled with concrete made at 73° F., sealed with a glass plate and heavy grease, and placed in a metal can 11 inches in diameter and 15 inches in height. Expanded mica was placed around all surfaces of the cylinder mold and on the glass plate, and the cans were stored in laboratory air maintained at a temperature of 73° F. Each cylinder was molded with a copper-constantan thermocouple at its symmetrical center and the temperatures were recorded on an eight-point potentiometer. The amount of each retarder used was that amount necessary for 2½ to 3 hours retardation at 73° or 90° F.

All cylinders for compressive strength tests were capped on both ends with high-alumina cement after removal from the molds, and at least 48 hours prior to testing. All caps were within 0.001 inch of planeness. The cylinders were loaded at a rate of 35 p.s.i. per second.

Beams for flexural strength were tested in accordance with AASHO method T 97. When bearings on beams did not meet the requirements for planeness, they were ground with a power driven carborundum wheel.

Beams measuring 3 by 4 by 16 inches were tested for resistance to freezing and thawing in accordance with the standard method of fast freezing and thawing in water, AASHO method T 161.

Measurements for volume change of 3- by 4- by 16-inch beams were made on a horizontal comparator with a micrometer dial reading to 0.0001 inch at 1 end and a micrometer barrel reading to 0.0001 inch at the other end. Beams were molded with stainless steel studs in the ends and stored with the 4-inch axis in a vertical position during the curing and drying period.

Tests for static modulus of elasticity were made on 6- by 12-inch cylinders at 7 and 28 days by an autographic stress-strain recorder with a 6-inch gage length. These tests were paralleled by sonic modulus of elasticity tests on 6- by 6- by 21-inch beams which were made from the same batch of concrete as the cylinders.

Density and absorption tests were made on 2 disks, each 1 inch in thickness, which were cut from the top, middle, and bottom of the 6- by 12-inch concrete cylinders. The cylinders were molded from mixes Nos. 1 and 2. The disks were cut by a diamond wheel at an age of 28 days from moist-cured cylinders and were weighed in air and under water. They were then dried to constant weight in a forced air oven at 190-200° F.

One procedure in the mixing and testing of the plastic concrete and the testing of hardened concrete should be emphasized. Every

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[4] *Measuring the rate of hardening of concrete by bond pullout pins*, by T. M. Kelly and D. E. Bryant, ASTM Proceedings, vol. 57, 1957, pp. 1029-1042. This method is referred to as the pin pullout method.

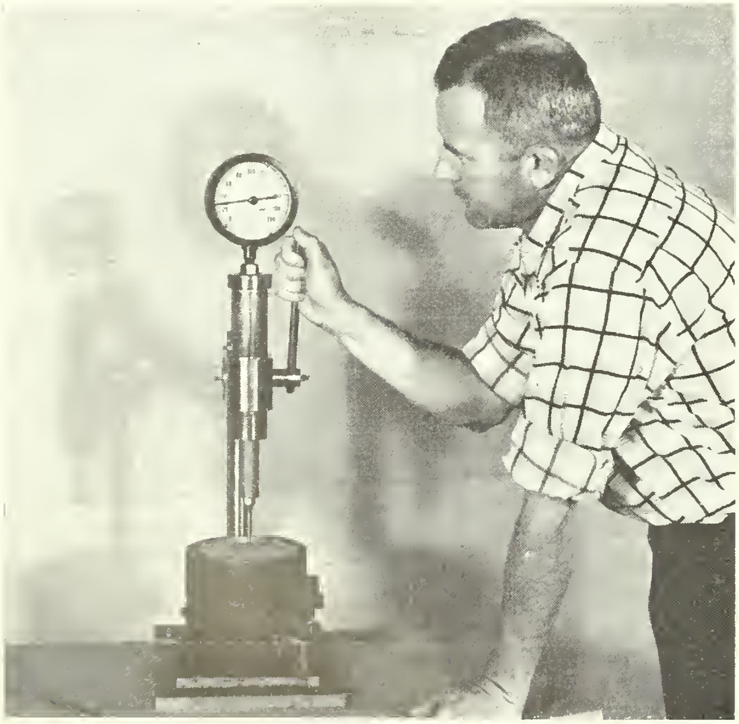


Figure 1.—Proctor penetration device.

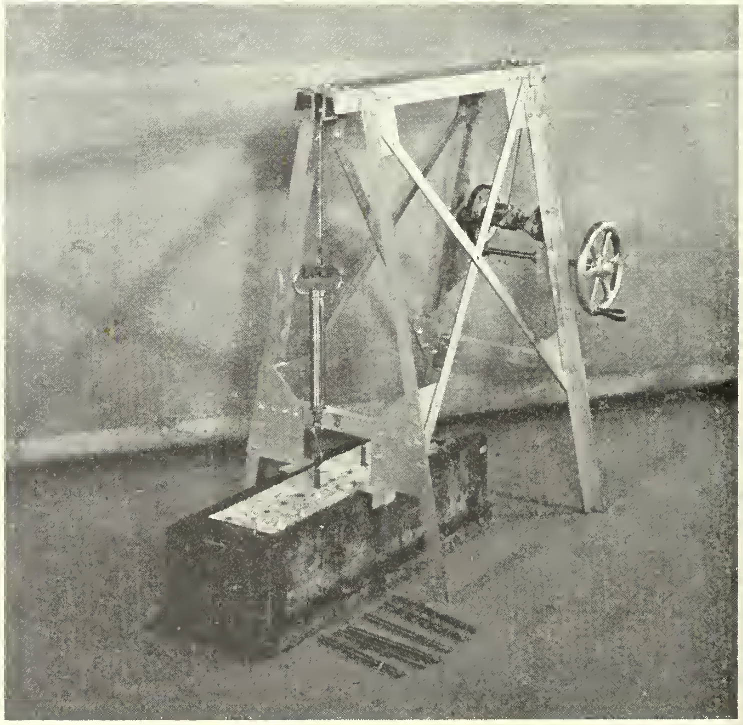


Figure 2.—Pin pullout device.

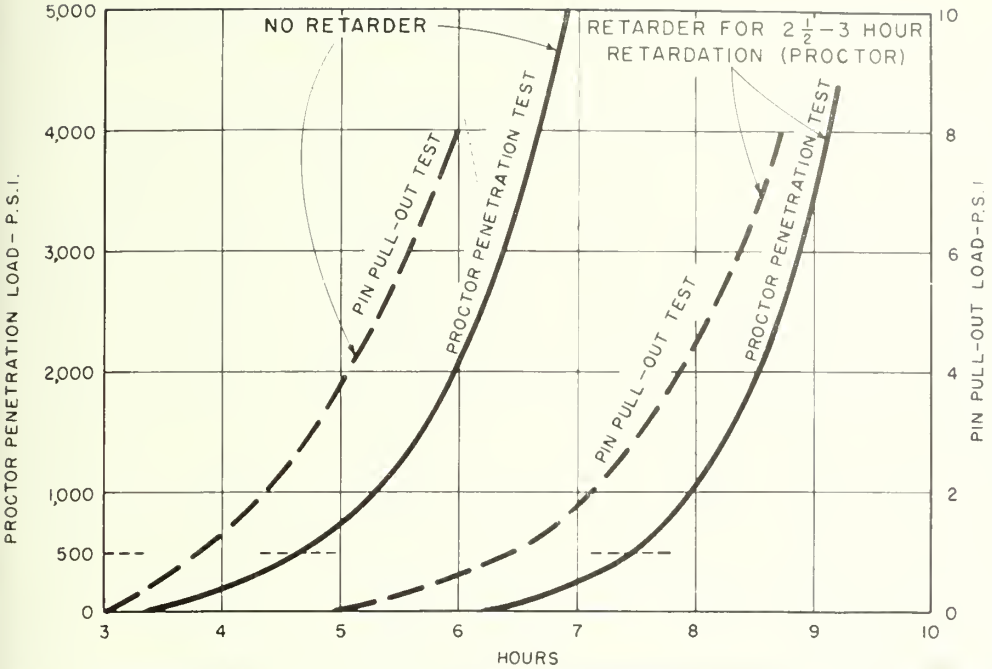


Figure 3.—Typical curve for Proctor penetration and pin pullout tests for retardation time on concrete.

test made on plastic or hardened concrete included specimens from mix No. 1, the reference mix without retarder.

Discussion of Test Results

The test data results are shown in tables 4–13, and in figures 4–12. The effect of retarders on the concrete are discussed in terms of the main objectives of the study, which were to determine the effect of the retarder on: (1) retardation of setting time, (2) water content and air-entrainment, (3) strength, (4) durability, and (5) volume change. Additional tests, such as the effect of overdosage of retarders, retardation and strength at elevated temperature, modulus of elasticity, delayed vibration, density and absorption, and temperature rise, are also discussed.

Effect of Admixtures on Time of Retardation

The amount of each retarder and the resulting retardation, as measured by the Proctor penetration test at 500 p.s.i. or 4,000 p.s.i. penetration pressure, or by the pin pullout test at 8 p.s.i. are shown in table 4. Each value is the average of 2 tests that were made on 2 days. Retardation is the difference in setting time between that for the reference mix (No. 1) and that for the concrete with the retarder (mix No. 2). Recommendations for the amount of retarder to be used were furnished by the manufacturers of 22 of the retarders included in these tests. For 13 retarders, the amount recommended was found to give the desired retardation of 2½ to 3 hours. For two retarders, 80 percent of the recommended amount was found to be sufficient, and for three other retarders, 120 to 125 percent of the recommended amount was needed. For the other 4 retarders, 150 to 500 percent of the recommended amount was required to obtain an effective amount of retardation. These 4 were retarders Nos. 2, 6, 22, and 23. Except for sample No. 6, these retarders were lignosulfonates.

The 500 p.s.i. penetration pressure of the Proctor test is believed to indicate the initial setting of the mortar used. At the initial setting, however, the mortar could still be remolded without injury to its strength. When determinations were made using a penetration pressure of 4,000 p.s.i., a condition at or near the final setting of the mortar was indicated. Attempts to rework the mortar at that time resulted in its disruption. In the pin pullout test, no attempt was made to define or determine the setting time of the concrete. Conducted under defined conditions, the test was intended to permit comparisons of the rates of hardening of different concretes. The comparison of typical retardation values for the Proctor penetration and pin pullout tests, shown in figure 3, indicated that a 500 p.s.i. penetration corresponds to a pin pullout load of 2½ to 2¾ p.s.i. However, the ranges of values shown in figure 4 indicated that for the concretes used in these tests, the Proctor penetration test with either of the loads used appeared to be more suitable for measuring setting time and retardation than the pin pullout test. The wide range of values for the pin pullout test and lack of a definite central point indicated that the test result probably was affected by uncontrolled variables to a greater extent than the Proctor penetration test.

Effect of Retarders in Concrete

Table 5 shows the amount of Vinsol resin solution (air entraining agent) used in concretes for mixes Nos. 2 and 3 for strength tests at 73° F. and the resulting water and air contents of the concrete. (The retarders generally were given their identification numbers in ascending order of the reduction in water requirements for mix No. 2. The exception is retarder No. 12 which is slightly out of place in the table.) The detailed data for the reference mix (No. 1) are not tabulated for results on each retarded concrete mix, but the average values for all reference mixes are given in a footnote. The values given in table 5 are averages of five rounds of tests.

The reduction in water content shown for mix No. 2 is the difference between the amount of water used for mix No. 1 and mix No. 2 made on the same day. This is shown as a percentage of the amount of water used for

(Continued on page 142)

Table 1.—Retardation time for concrete mix No. 2 at 73° F.[1]

Retarder No.	Amount retarder used	Air [2]	Slump	Proctor penetration test,[3] retardation at—		Pin pull-out test, retardation at 8 p.s.i.[4]
				500 p.s.i.[4]	4,000 p.s.i.[4]	
	Oz./bag	*Percent*	*Inches*	*Hr. Min.*	*Hr. Min.*	*Hr. Min.*
1	1.1	5.0	2.6	2:50	2:45	2:40
2	14.4	5.0	2.5	2:30	2:10	2:20
3	4.0	6.1	3.0	2:45	2:35	2:30
4	3.0	5.4	3.2	2:30	2:30	2:55
5	4.0	5.4	3.1	2:40	2:20	1:25
6	4.0	5.7	3.2	2:20	2:15	2:30
7	3.5	5.0	2.7	2:45	2:40	3:00
8	12.0	6.3	3.2	2:50	3:00	2:25
9	2.8	6.0	2.8	2:30	2:35	2:15
10	3.2	6.2	2.6	2:30	2:45	2:20
11	4.8	5.4	3.0	2:50	2:30	2:50
12	4.0	5.2	3.0	2:50	2:50	2:05
13	4.8	5.4	3.0	2:20	2:30	2:15
14	5.0	4.3	3.3	2:40	2:45	2:40
15	4.0	7.5	3.0	2:35	3:00	1:55
16	4.8	6.2	3.0	2:50	2:35	1:50
17	3.2	5.5	2.6	2:50	3:00	2:20
18	4.0	5.0	2.8	2:20	2:00	1:00
19	4.0	5.5	2.8	2:40	3:00	2:05
20	9.0	5.0	2.2	2:25	2:30	2:35
21	9.0	6.4	3.5	2:30	2:40	2:10
22	4.0	10.0	2.6	2:30	2:30	1:50
23	6.4	7.0	3.0	2:40	2:40	1:40
24	9.6	5.4	2.4	2:40	2:25	1:55
25	4.0	7.0	2.8	2:35	2:35	1:30

[1] Each value is an average of 2 tests made on different days.

[2] Neutralized vinsol resin solution used when needed.

[3] A load of 500 p.s.i. indicates the vibration limit of the concrete; a load of 4,000 p.s.i. indicates hardened concrete. Retardation is the delay in time of hardening of the concrete containing the retarders as compared with non-retarded concrete made on the same day.

[4] Time for nonretarded concrete to reach: Proctor penetration load of 500 p.s.i. is 5 hrs. 10 min. (±20 min.); Proctor penetration load of 4,000 p.s.i. is 7 hrs. 35 min. (±30 min.); pin pullout load of 8 p.s.i. is 6 hrs. 40 min. (±20 min.).

STATE LEGAL MAXIMUM LIMITS OF MOTOR VEHI

Prepared by

Line	State	Width inches[1]	Height ft.-in.	Length—feet[2]				Number of towed units[3]			Axle load—pounds				Type of restri
				Single unit		Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Single		Tandem		
				Truck	Bus						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance	
1	Alabama	96	[6] 12-6	35	40	50	NP	1	NP	NP	18,000	19,800	36,000	39,600	Table
2	Alaska	96	12-6	35	[19] 40	60	60	1	1	2	18,000		32,000		Table-tire cap
3	Arizona	96	13-6	40	40	65	65	1	1	2	18,000		32,000		Table
4	Arkansas	96	13-6	35	40	50	50	1	1	NP	18,000	[7] 18,500	32,000	32,500	Spec. maximum
5	California	96	13-6	35	[9] 35	60	65	NR	NR	NR	18,000		32,000		Table
6	Colorado	[11] 96	[12] 13-6	35	40	60	60	1	2	2	18,000		36,000		Formula-spec.
7	Connecticut	102	12-6	50	50	50	NP	1	NP	NP	22,400	22,848	36,000	36,720	Spec. lim.-tire
8	Delaware	96	[6] 12-6	40	42	50	60	1	1	2	20,000		36,000		Table-spec. lim [13]
9	District of Columbia	96	12-6	35	35	50	80	1	1	NP	22,000		38,000		Table
10	Florida	96	[6] 12-6	[14] 35	40	50	50	1	1	NP	20,000	22,000	40,000	44,000	Table
11	Georgia	96	13-6	[15] +39	[15] +45	50	50	1	1	NP	18,000	20,340	36,000	40,680	Spec. maximum
12	Hawaii	108	13-0	40	40	55	65	1	1	2	24,000		32,000		Formula[17]
13	Idaho	[18] 96	14-0	35	[19] 40	60	65	1	1	2	[20] 18,000		[20] 32,000		Table[20]
14	Illinois	96	13-6	42	42	50	50	1	1	2	[21] 18,000		32,000		Spec. lim.-tire
15	Indiana	96	13-6	36	40	50	50	1	1	2	[23] 18,000	[23] 19,000	[23] 32,000	[23] 33,000	Spec. lim.-tire
16	Iowa	96	13-6	35	[19] 40	50	NP	1	[24] 1	NP	18,000	18,540	32,000	32,960	Table
17	Kansas	96	13-6	35	[19] 40	50	50	1	1	NP	18,000		32,000		Table
18	Kentucky	96	[25] 13-6	[26] 35	[26] 35	[27] 50	NP	1	NP	NP	18,000	[28] 18,900	32,000	[28] 33,600	Spec. lim.-tire [29]
19	Louisiana	96	13-6	35	[19] 40	50	60	1	1	NP	18,000		32,000		Axle lim.-tire
20	Maine	96	[30] 12-6	50	50	50	50	1	1	NP	[30] 22,000		[30] 32,000		Table-tire cap.
21	Maryland	96	[6, 45] 12-6	55	55	55	55	NR	NR	NR	22,400		40,000		Formula
22	Massachusetts	96	NR	35	[19] 40	[31] 50	NP	1	NP	NP	22,400		36,000		Table-spec. l
23	Michigan	96	13-6	35	40	55	55	1	1	2	[33] 18,000		[34] 32,000		Axle lim.-tire
24	Minnesota	96	13-6	40	40	50	50	1	1	NP	18,000		32,000		Table
25	Mississippi	96	[6] 12-6	35	40	[36] 50	50	1	1	NP	18,000		28,650	[35] 32,000	Table-tire cap
26	Missouri	96	12-6	35	40	50	50	1	1	2	18,000		32,000		Table
27	Montana	[18] 96	13-6	35	40	60	60	1	1	[37] 2	18,000		32,000		Table
28	Nebraska	96	13-6	40	40	60	60	1	1	2	18,000	18,900	32,000	33,600	Table
29	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	Table
30	New Hampshire	96	13-6	35	[35] 40	50	50	NR	NR	NR	22,400		36,000		Tables-spec.
31	New Jersey	96	13-6	35	[39] 35	50	[40] 50	1	1	NP	22,400	23,520	32,000	33,600	Spec. limits
32	New Mexico	[41] 96	13-6	40	40	65	65	1	1	2	21,600		34,320		Table
33	New York	96	13-0	35	[42] 35	50	50	1	1	NP	22,400		36,000		Formula
34	North Carolina	96	[6] 12-6	35	[19] 40	[43] 50	[43] 55	1	1	NP	18,000	19,000	36,000	38,000	Spec. limits
35	North Dakota	96	13-6	[14] 35	[19] 40	60	60	1	1	2	18,000		32,000		Formula
36	Ohio	96	13-6	35	[19] 40	50	60	1	NR	NR	19,000		31,500		Formula
37	Oklahoma	96	13-6	35	45	50	50	1	1	NP	18.000		32,000		Table
37	Oregon	96	[45] 13-6	35	[35] 40	[46, 35] 55	[35] 65	1	1	[35] 2	[47] 18,000		[47] 32,000		Table[48]
38	Pennsylvania	96	[6] 12-6	35	[19] 40	50	[40] 50	1	1	NP	22,400	23,072	36,000	37,080	Spec. limits
39	Puerto Rico	96	12-6	35	35	50	50	1	1	NP	NS		NS		Spec. lim.-tire
40	Rhode Island	102	12-6	40	40	50	50	1	1	NP	22,400		NS		Spec. limits
42	South Carolina	96	13-6	[14] 35	[19] 40	55	60	1	1	NP	20,000		32,000		Table
43	South Dakota	96	13-0	35	40	50	60	1	1	2	18,000		32,000		Table
44	Tennessee	96	[6] 12-6	35	40	50	50	1	[53] 1	NP	18,000		32,000		Table
45	Texas	96	13-6	35	40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table
46	Utah	96	14-0	45	45	60	60	NR	NR	NR	18,000		33,000		Table
47	Vermont	96	12-6	50	50	50	50	1	1	NP	NS		NS		Spec. lim.-tire
48	Virginia	96	[6] 12-6	35	[35] 40	50	50	1	1	NP	18,000		[57] 32,000		Table
49	Washington	96	13-6	35	[19] 40	60	[58] 65	1	1	[58] 2	18,000	[59] 18,500	32,000	[59] 33,000	Table-spec. l
50	West Virginia	96	[6] 12-6	35	[19] 40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table
51	Wisconsin	96	13-6	35	40	50	50	1	1	NP	18,000	[60] 19,500	30,400	32,000	Table-formula
52	Wyoming	96	13-6	40	40	60	60	1	1	2	18,000		32,000	[62] 36,000	Table
	AASHO Policy	96	12-6	35	[19] 40	50	60	1	1	NP	18,000		32,000		Table
Number of States	Higher	3	33	17	28	18	9	5	7	24	31		30		Formula 6
	Same	49	19	35	19	34	12	47	41	28	21		21		Table 32
	Lower	0	0	0	5	0	31	0	4	0	0		1		Specified lim 14

NP—Not permitted. NR—Not restricted. NS—Not specified.

[1] Various exceptions for farm and construction equipment; public utility vehicles; house trailers; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles; for safety accessories, on designated highways, and as administratively authorized.

[2] Various exceptions for utility vehicles and loads, house trailers and mobile homes.

[3] When not specified, limited to number possible in practical combinations within permitted length limits; various exceptions for farm tractors, mobile homes, etc.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhang of 3 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerances as applicable.

[6] Auto transports 13 feet 6 inches.

[7] Does not apply to combinations of adjacent load-carrying single axles.

[8] 56,000 pounds on load-carrying axles, exclusive of steering-axle load.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 45 feet without restriction.

[10] Effective September 18, 1959 on Interstate, 4-lane, and designated State highways.

[11] Buses 102 inches.

[12] On designated highways; 12 feet 6 inches on other highways.

[13] Legal limit 60,000 pounds, axle spacing 27 feet or more.

[14] Three-axle vehicles 40 feet.

[15] Truck 39.55 feet; bus 45.20 feet.

[16] 63,280 pounds maximum, except on roads under

[17] 700 (L +40) when L is 18' or less; 800 (L +40) with span of 20' or over.

[18] Buses 102 inches on highways of surfaced wid

[19] Less than three axles 35 feet.

[20] Special limits for vehicles hauling timber and including livestock; single axle 18,900 pounds, tande mitted 66,000 pounds maximum at 21-foot axle spacin foot axle spacing.

[21] On designated highways; 16,000 pounds on oth

[22] Without tandem axles 45,000 pounds.

[23] On designated highways; single axle 22,400 po excesses of weight under one or more limitations of

[24] Limited to 4 wheel trailer towed by truck not ex

[25] Class AA highways; 12 feet 6 inches on other

[26] On designated highways; trucks 26.5 feet and

[27] Class AA highways; 45 feet on other highways.

[28] Class AA highways only.

[29] Maximum gross weight on Class A highways 42

[30] Including load 14 feet; various exceptions for

[31] Effective September 21, 1959.

[32] Subject to axle and tabular limits.

[33] Single axle spaced less than 9 feet from neares

[34] On designated highways only and limited to an

[35] On designated highways only (by permit in Or

[36] Auto transports permitted 50 feet.

[37] Semitrailer and semitrailer converted to full tra

WEIGHTS COMPARED WITH AASHO STANDARDS

s, July 1, 1960

	Applicable to:		Specified maximum gross weight—pounds[4]						Practical maximum gross weight—pounds[5]						Line
			Truck		Truck-tractor semitrailer			Other combination	Truck		Truck-tractor semitrailer			Other combination	
	Any group of axles	Total wheel base only	2-axle	3-axle	3-axle	4-axle	5-axle		2-axle	3-axle	3-axle	4-axle	5-axle		
		X							27 800	47,600	47,600	60,010	64,650	NP	1
	nder 18'	Over 18'	29,000	43,000	47,000	61,000	75,000	75,000	26,000	40,000	44,000	58,000	72,000	75,000	2
	nder 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	3
									26,500	40,500	45,000	59,000	65,000	65,000	4
	nder 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	5
0).		X	30,000	46,000					26,000	44,000	44,000	62,000	76,000	76,000	6
			32,000	50,000	50,000	60,000	60,000	NP	30,848	44,720	51,000	61,200	61,200	NP	7
	X		30,000	46,000	48,000	60,000	60,000	60,000	28,000	48,000	48,000	56,350	60,000	60,000	8
	X								30,000	46,000	52,000	[54] 58,450	[54] 61,490	[54] 64,650	9
		X							30,000	52,000	52,000	65,200	73,095	73,095	10
								63,280	28,340	48,680	48,680	63,280	63,280	63,280	11
	X								32,000	38,800	56,000	64,000	72,000	80,000	12
	X								26,000	40,000	44,000	58,000	72,000	76,800	13
			36,000	[22] 41,000	45,000	59,000	72,000	72,000	26,000	40,000	44,000	58,000	72,000	72,000	14
								72,000	27,000	41,000	45,000	59,000	[23] 73,000	[23] 73,000	15
	X								26,540	40,960	45,080	59,500	73,280	NP	16
	X								26,000	40,000	44,000	55,470	63,890	63,890	17
			36,000	50,000	54,000	59,640	73,280	NP	26,900	41,600	45,800	59,640	73,280	NP	18
									26,000	40,000	44,000	58,000	72,000	76,000	19
	X		32,000	[30] 50,000	50,000	60,000	60,000	60,000	30,000	40,000	50,000	60,000	60,000	60,000	20
0)		X			65,000	65,000	65,000	65,000	30,400	48,000	52,800	65,000	65,000	65,000	21
5)		X	[32] 46,000	[32] 60,000	[32] 60,000	[32] 60,000	[32] 60,000	NP	30,400	44,000	52,800	60,000	60,000	NP	22
									26,000	[35] 40,000	44,000	[35] 58,000	[35] 66,000	[35] 102,000	23
	X								26,000	40,000	44,000	58,000	68,000	72,500	24
	X								26,000	[35] 40,000	44,000	59,000	[35] 64,650	[35] 64,650	25
	X								26,000	40,000	44,000	55,470	64,650	64,650	26
	nder 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	27
	X		36,000	54,000	54,000	71,146	71,146	71,146	26,780	41,200	45,320	59,740	73,280	73,280	28
	nder 18'	Over 18'							26,900	41,600	45,800	60,500	75,200	76,800	29
		X	33,400	[38] 47,500	52,800	66,400			30,400	44,000	52,800	66,400	66,400	66,400	30
			30,000	40,000	60,000	60,000	60,000	60,000	31,500	41,600	55,040	63,000	63,000	63,000	31
	nder 18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	32
L		X					65,000	65,000	30,400	44,000	52,800	65,000	65,000	65,000	33
			31,500	46,200	46,200	65,100	65,100	65,100	27,000	46,000	46,000	65,100	65,100	65,100	34
0)	nder 18'	Over 18'							26,000	38,000	44,000	56,000	[44] 64,000	[44] 64,000	35
L		X							27,000	39,500	46,000	58,500	71,000	78,000	36
	X								26,000	40,000	44,000	58,000	72,000	73,280	37
	nder 18'	Over 18'					[48] 74,000	[48] 76,000	26,000	40,000	44,000	58,000	[48] 72,000	[48] 76,000	38
			33,000	47,000	50,000	60,000	60,000	62,000	31,072	45,080	51,500	61,800	61,800	63,860	39
															40
			[50] 36,000	[51] 44,000	[52] 50,000	[13] 60,000	60,000	88,000	30,400	44,000	50,000	60,000	60,000	88,000	41
	X								28,000	40,000	48,000	60,000	66,839	71,115	42
	X								26,000	40,000	44,000	58,000	72,000	73,280	43
		X							26,000	40,000	44,000	58,000	61,580	43,500	44
	X								26,900	41,600	45,800	60,500	75,200	75,600	45
	nder 18'	Over 18'							26,000	41,000	44,000	59,000	74,000	79,900	46
			30,000	[55] 50,000	50,000	[56] 60,000	[56] 60,000	[56] 60,000	30,000	[55] 50,000	50,000	60,000	60,000	60,000	47
	X						[35] 56,800	[35] 56,800	26,000	40,000	44,000	56,800	56,800	56,800	48
	nder 18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	49
	X								26,900	41,600	45,800	57,844	63,840	63,840	50
6)	X								27,500	40,000	47,000	59,500	73,000	73,000	51
	X								26,000	44,000	44,000	62,000	73,950	73,950	52
L[2]	X								26,000	40,000	44,000	55,470	61,490	71,900	
	19	19							29 22 0	27 20 4	29 22 0	48 3 0	42 2 7	26 0 25	

6, pounds maximum.

3'; 0 (L+40) on highways having no structures

as administratively authorized.

rates, aggregates, and agricultural products
weight table: vehicle with 3 or 4 axles per-
s permitted 79,000 pounds maximum at 43-

0 nds; tolerance of 1,000 pounds on total of all
ht
t.

s.

ways 30,000 pounds.
ts and construction materials.

ds.
otherwise 26,000 pounds.

[38] Dual-drive axles; otherwise 40,000 pounds.

[39] Or as prescribed by P.U.C.

[40] Exception for poles, pilings, structural units, etc., permitted 70 feet.

[41] On designated highways 102 inches.

[42] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.

[43] Including front and rear bumpers.

[44] Approved equipment 73,280 pounds.

[45] Certain types of vehicles and commodities under annual permits on designated highways up to 13 feet 6 inches

[46] 60 feet allowed truck tractor semitrailer on major routes designated by permit.

[47] Logging vehicles permitted 3-foot wheelbase tolerance, 19,000-pound single axle, 34,000-pound tandem axle.

[48] Governs gross weight permitted on highways designated by resolution of State highway commission or by permit.

[49] Single unit truck with 4 axles permitted 60,000 pounds.

[50] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.

[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.

[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 50,000 pounds.

[53] Limited to 3,500 pounds.

[54] Pavements only, maximum legal load for bridges 56,800 pounds.

[55] On Interstate routes 40,000 pounds.

[56] Tandem axles on trailer equipped with adequate brakes.

[57] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.

[58] Three-unit combinations on designated highways.

[59] Vehicles hauling logs permitted wheelbase and gross weight tolerances. Discretionary enforcement tolerances not included in computation of practical maximum gross weights.

[60] Axle load 21,000 pounds on 2-axle trucks hauling peeled or unpeeled forest products cut crosswise.

[61] On Class A and Class B highways. All axles of a vehicle or combination—73,000 pounds maximum.

[62] Based on ruling of Attorney General.

[63] Dimensional limits effective August 20, 1960; weight limits to be established by administrative regulations.

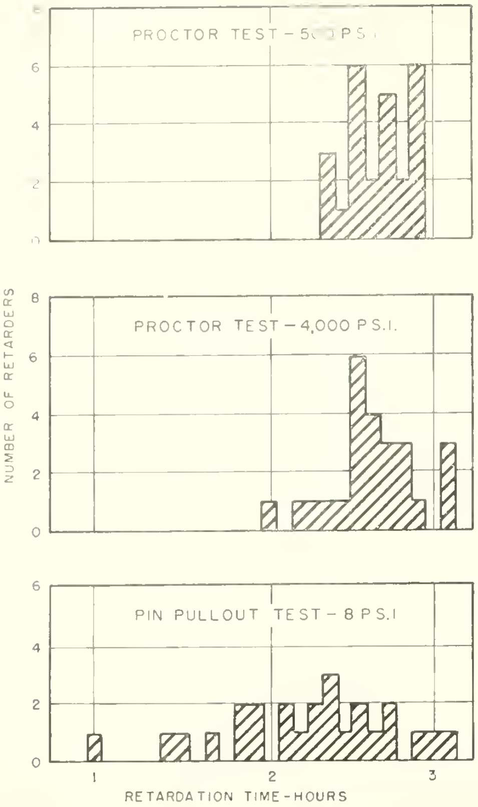


Figure 4.—Time distribution of retardation.

(Continued from page 139)

mix No. 1. A comparison between this reduction in water content and the amount of retarder used for mix No. 2 is shown in figure 5. It was noted that some retarders were much more effective in permitting a reduction in the amount of water required to obtain concrete of a given slump. A comparison between the amount of reduction in water and the amount of retarder used per bag of cement showed that retarder No. 25 was the most effective and retarder No. 2 was the least effective. These retarders reduced the water required for mix No. 2 by 2.9 percent and 0.2 percent, respectively, per ounce of retarder used per bag of cement. The amount of volatile organic material used is shown in table 1 and figure 5. For the lignosulfonates this amount was between two and four ounces per bag except for retarders Nos. 2 and 24. Of the 9 most effective retarders with respect to reduction in water, all were lignosulfonates except for 1 organic acid.

Water content

The concretes of mix No. 3 were prepared with cement contents reduced in accordance with the amount of reduction in water found possible for mix No. 2, the objective being to have the same water content in mix No. 3 as in mix No. 1. For retarders which permitted a reduction in water of 5 percent or less in mix No. 2, the cement content for mix No. 3 was reduced 0.25 bag per cubic yard. (This is designated as group A in table 3.) For retarders which permitted a reduction of water of over 5 percent but less than 10 percent, the cement content for mix No. 3 was reduced 0.5 bag per cubic yard (group B in table 3). For retarders permitting a reduction in water of 10 percent or more, the cement content was reduced 0.75 bag per cubic yard (group C). It was found that, due to uncontrollable variables, it was not possible to prepare all batches of concrete with these exact cement contents, and at the same time maintain a slump of 2.8 ± 0.3 inches and use the same amount of water in mix No. 3 as was used in the reference mix. To hold the cement content and slump as close as possible to the desired values, slight variations in the water content were permitted. For 18 of the 25 retarders, the water contents were within 0.1 gallon per bag of cement of the desired values, and in only one case (sample No. 23) did the actual water content and the desired content differ by more than 0.2 gallon per bag. In view of these small differences, mix No. 3 was considered to have the same water content as the reference mix.

Air entraining agent

The amount of Vinsol resin solution required to entrain 5 to 6 percent air in the mix No. 2 concrete prepared with an organic acid retarder (Nos. 4, 5, 6, 8, or 9) was approximately two-thirds of that used in the reference mix. The water reduction caused by the five organic acid retarders varied from 5.3 to 6.6 percent. The amount of air-entraining solution used in the mix No. 3 concrete with a decreased cement content of ¼ to ¾ of a bag per cubic yard was approximately the same as the quantity needed for mix No. 2. Although experience has shown that the organic acid retarders cause no air entrainment in concrete, these data indicate that they aid air entrainment and permit a moderate reduction in water content when used at the rate of 2.8 to 12 ounces per bag of cement.

The concrete prepared with each of the lignosulfonate retarders showed a wide range, 2.8 to 11.8 percent, in water reduction. The reduction of 2.8 percent was obtained by use of a powdered lignosulfonate which contained over 95 percent of an inorganic filler. Two lignosulfonates permitted reductions of 5.2 and 5.9 percent, respectively; 12 permitted reductions of 7 to 10 percent; and 4 permitted reductions of 10 to 12 percent. It should be noted that in retarder samples Nos. 1 and 2, although the water reduction for concrete was lower than is usually specified, satisfactory retardation was obtained.

Air content

The effect of each of the 25 retarders on air content of concrete and the amount of air-entraining admixture used is shown in figure 6.

Table 5.—Mix data for strength tests [1]

Retarder No.	Water for mix No. 1 [2]	Mix No. 2—same cement content as mix No. 1						Mix No. 3—same water content as mix No. 1				
		AEA added	Cement	Slump	Air	Water	Reduction in water [3]	AEA added	Cement	Slump	Air	Water
	Gal./bag	*Ml./bag*	*Bags/cu. yd.*	*Inches*	*Percent*	*Gal./bag*	*Percent*	*Ml./bag*	*Bags/cu. yd.*	*Inches*	*Percent*	*Gal./bag*
1	5.75	17.1	6.0	2.8	5.0	5.69	1.1	19.0	5.7	2.7	5.6	5.89
2	5.77	16.2	6.0	2.6	5.2	5.61	2.8	18.0	5.7	2.9	5.6	5.93
3	5.79	--------	6.0	3.1	5.9	5.49	5.2	--------	5.5	2.8	5.6	5.89
4	5.81	13.7	6.0	2.9	5.7	5.50	5.3	14.5	5.4	2.8	5.8	6.01
5	5.74	11.5	6.0	3.1	5.5	5.41	5.7	12.3	5.5	2.7	5.8	5.81
6	5.87	15.1	6.0	2.7	5.4	5.53	5.8	16.4	5.5	2.9	5.9	6.03
7	5.76	8.3	6.0	2.7	5.4	5.42	5.9	9.2	5.5	2.6	5.5	5.84
8	5.78	15.8	6.0	2.6	5.5	5.43	6.1	14.7	5.5	2.8	5.6	5.88
9	5.91	14.5	6.0	2.9	5.2	5.52	6.6	16.6	5.5	2.6	5.7	6.04
10	5.86	9.4	6.0	3.0	5.4	5.44	7.2	8.6	5.6	2.7	5.1	5.76
11	5.71	3.7	6.0	2.7	5.7	5.29	7.4	4.1	5.5	2.7	5.5	5.71
12	5.86	2.5	6.0	2.6	5.4	5.37	8.4	--------	5.5	2.7	5.6	5.87
13	5.77	2.6	6.0	2.7	5.7	5.30	8.1	1.3	5.5	2.6	5.7	5.75
14	5.67	2.2	6.0	3.0	5.7	5.21	8.1	2.8	5.5	2.7	5.8	5.72
15	5.73	--------	6.0	3.0	6.8	5.26	8.2	--------	5.5	2.8	7.0	5.67
16	5.67	4.4	6.0	2.9	6.1	5.20	8.3	3.7	5.5	2.8	5.8	5.74
17	5.79	11.5	6.0	2.8	5.5	5.31	8.3	11.3	5.5	2.9	5.5	5.81
18	5.77	4.5	6.0	2.7	5.6	5.27	8.7	4.4	5.5	2.6	5.7	5.75
19	5.74	7.2	6.0	2.5	5.7	5.24	8.7	6.4	5.5	2.6	5.4	5.78
20	5.86	--------	6.0	2.8	5.3	5.30	9.6	1.8	5.5	2.6	5.5	5.81
21	5.84	4.0	6.0	2.7	5.7	5.27	9.8	2.8	5.5	2.6	5.7	5.72
22	5.80	--------	6.0	3.2	10.0	5.22	10.0	--------	5.2	3.1	10.0	5.88
23	5.75	--------	6.1	2.6	5.8	5.15	10.4	--------	5.3	2.6	5.4	6.03
24	5.78	--------	6.0	2.6	5.9	5.13	11.2	--------	5.2	2.6	5.8	5.86
25	5.76	--------	6.0	2.8	7.6	5.08	11.8	--------	5.2	2.8	9.5	5.72

[1] Each value is an average of five tests made on different days. Amount of retarder used is the same amount as shown in table 4.

[2] Water content for mix No. 1 is water used in reference mix made on same day as retarded concrete mixes. Average values for mix No. 1 are: air-entraining agent (AEA) added, 20 ml./bag; cement, 6.0 bags/cu. yd.; slump, 2.9 inches; air, 5.4 percent.

[3] Average reduction in water-cement ratio as compared with that required for the reference mix made on the same day.

Seven of the 19 lignosulfonate retarders, vithout the use of an air-entraining solution, ntrained more than 5 percent of air in the oncrete. Three of these retarders caused air ontent of about 6 percent, while the maxinum air content, 10 percent calculated ravimetrically, was obtained for the concrete ontaining retarder No. 22.

The amount of air entraining solution ieeded with each of the other 12 lignosulonates to entrain 5 to 6 percent air in conrete mix No. 2 varied from about 1/8 to 1/2 of he quantity used in the reference mix, except n the case of retarder No. 2. The powdered xtender and the small amount of lignosulonate in this retarder probably accounted for he larger quantity of air-entraining solution equired.

Retarder No. 1, sucrose of the carbohydrate group, caused a small reduction (1.1 percent) n water content and required almost as much ir-entraining solution as the reference mix.

Effect of Admixtures on Strength of Concrete

Compressive strength test results

The results of the compressive strength ests on concrete containing the 25 retarders re given in table 6. Shown are the averages of five compressive strength tests for mixes Nos. 2 and 3 on 6- by 12-inch cylinders made on different days and tested at 3, 7, 28, and 365 days, and the ratios of these strengths to the average strengths of concrete without retarder (mix No. 1) made and tested on the same day as the concrete containing the retarder.

Average values for the compressive strength of all concretes prepared with and without the retarders are shown in figure 7. The curves show that both mix No. 2, with a reduced water content, and mix No. 3, with a reduced cement content, furnished higher average compressive strengths at all ages than was obtained without the use of retarders. This increase in compressive strength might be considered a secondary benefit obtained by the use of retarders, but it was a real and definite improvement in the characteristics of concrete.

As shown in table 6, all retarders except Nos. 22 and 25 furnished concrete of equal or higher strength at all ages than was obtained in the nonretarded concrete. Retarder No. 22 caused the entrainment of an excessive amount of air, and the strength suffered accordingly. Retarder No. 25 gave good results when used in mix No. 2 with a reduced water content. However, when used in mix No. 3 where the cement content was reduced, the concrete containing this retarder had lower strengths at all ages than the reference concrete. Retarder No. 25 also caused the entrainment of an excessive amount of air in mix No. 3. Study of the strengths obtained at all ages failed to show that any type of retarder gave better results than any other type.

Flexural strength test results

The results of the flexural strength tests for mixes Nos. 2 and 3 are given in table 7. This table gives the average of five tests on the 6- by 6- by 21-inch beams made on different days and tested at ages of 7 and 28 days. The ratios of these strengths to the strengths of concrete without retarder (mix No. 1) made and tested on the same day as the concrete containing the retarders are also given.

For some reason which was not apparent, the use of retarders failed to improve the flexural strength of concrete to the same extent as was found for the compressive strength. In the compression tests, the average strength of the reference mix concrete was 5,040 p.s.i. at an age of 28 days. The average strength of mix No. 2 retarded concrete was 115 percent of this, and that for mix No. 3 was 104 percent. In the tests for flexural strength, the reference concrete had

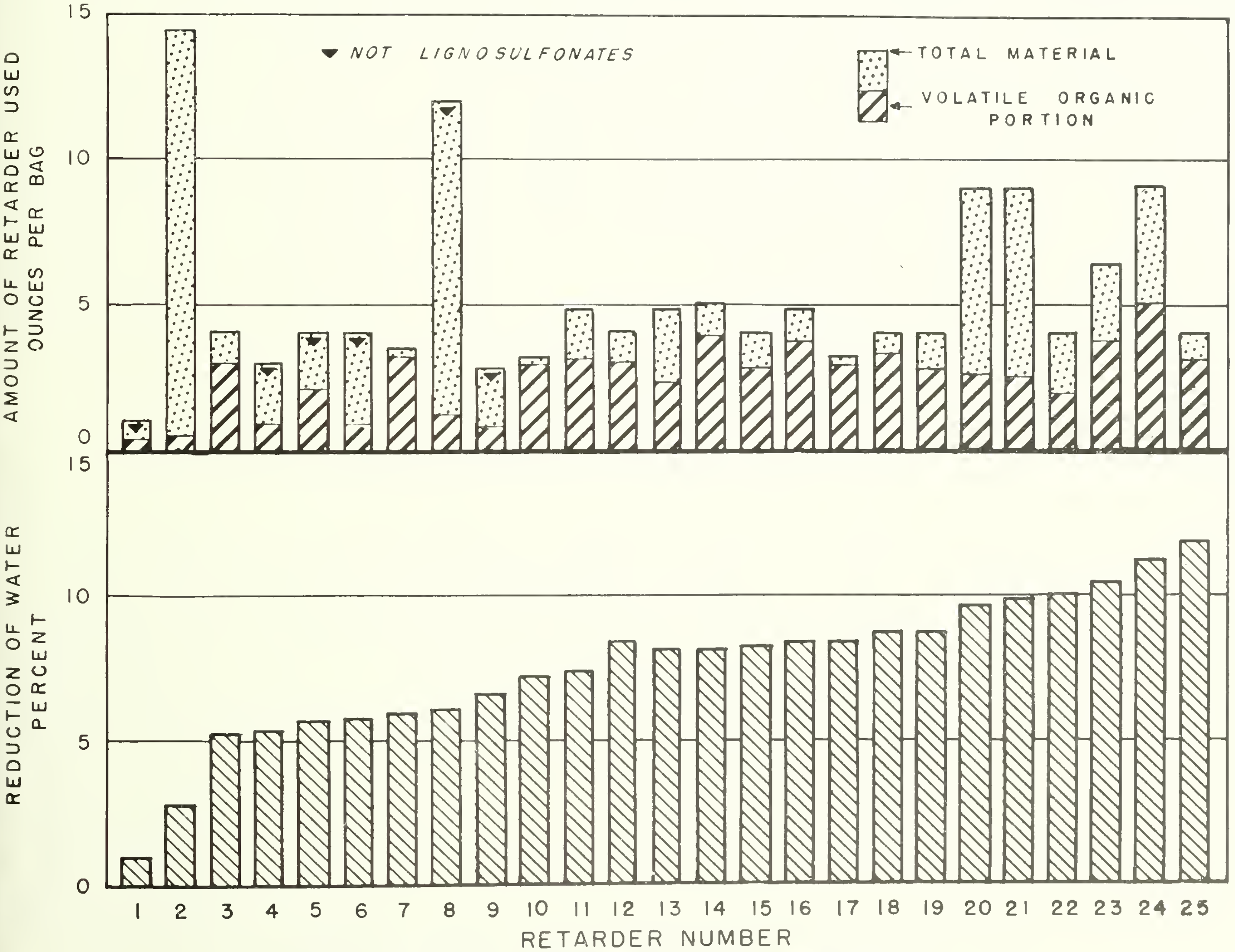


Figure 5.—Amount of retarder used and effect on water content.

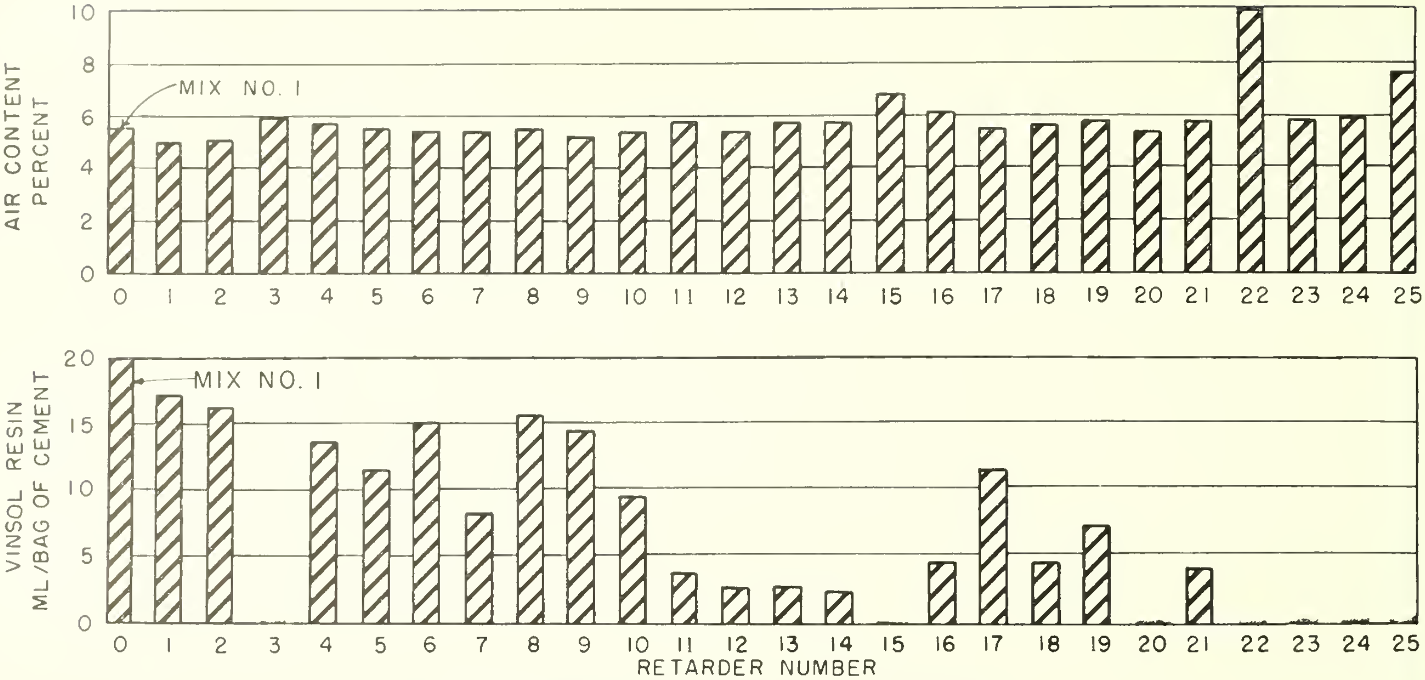


Figure 6.—Effect of retarder on air content of mix No. 2.

an average strength of 740 p.s.i. after 28 days, and mixes 2 and 3 of retarded concretes had average strengths of 104 and 97 percent, respectively, of the reference mix strength. It will be noted, however, that the ratio of the values of mixes Nos. 2 and 3 for compressive strength, 115 against 104, was practically the same as the ratio of values for flexural strength, 104 against 97. Consequently, it can be assumed that the results of the two sets of strength tests were valid and that the use of a retarder furnished concrete of lower relative flexural strength than compressive strength.

Studies of the different groups of retarders show that concretes prepared with the calcium lignosulfonate retarder had lower flexural strength than those prepared with the carbohydrate, the ammonium and sodium lignosulfonates, and the organic acids. No explanation can be given for this, nor for the behavior of concrete prepared with retarder No. 22. In the compression tests, concrete containing this retarder had the lowest strength, most likely due to the large amount of air entrained. In the flexural tests, this concrete had strengths lower than those of the reference concrete but with respect to the other retarded concretes, the strengths were reduced but slightly. It appears probable that in the acceptance testing of retarders, consideration should be given to the behavior of individual retarders and not assume that all retarders of a given type will affect concrete in the same manner.

In figure 8, average values of the effect of all retarders on the flexural strength of concrete are shown. These data are taken from two sets of specimens. The tests a ages of 7 and 28 days were made on 6- by 6- by 21-inch beams and the results obtained are shown in table 7. When these results were studied, it was observed that at an age of 28 days the average strength for the reference mix was greater than that for mix No. 3, and was approaching the strength fo mix No. 2. A determination of the relative strengths of these concretes at greater ages was believed desirable.

As no other beams of 6- by 6-inch cross section were available, plans were made to test for flexural strength by using the 3- by

Table 6.—Compressive strength tests [1]

Retarder No.	Compressive strength for mix No. 2 [2]								Compressive strength for mix No. 3 [2]							
	3 days		7 days		28 days		1 year		3 days		7 days		28 days		1 year	
	P.s.i.	Ratio	P.s.i.	Ratio	P.s.i.	Ratio	P.s.i.	Ratio	P.s.i.	Ratio	P.s.i.	Ratio	P.s.i.	Ratio	P.s.i.	Ratio
1	2,910	126	4,210	118	5,700	115	6,890	114	2,700	117	3,950	111	5,450	110	6,510	107
2	2,910	123	3,980	110	5,370	108	6,520	109	2,620	111	3,840	106	5,090	102	6,070	102
3	2,910	125	3,990	116	5,240	110	6,400	113	2,430	105	3,460	100	5,030	106	6,050	107
4	2,930	126	4,430	125	5,900	120	6,890	117	2,410	103	3,970	112	5,180	105	6,180	105
5	3,260	132	4,600	124	6,090	120	7,080	115	2,920	119	4,120	111	5,580	110	6,590	108
6	2,910	126	4,290	124	5,610	118	6,740	118	2,480	107	3,800	110	5,160	109	6,130	107
7	3,030	122	4,310	118	5,720	114	6,890	115	2,680	108	3,790	104	5,420	108	6,500	109
8	3,070	127	4,370	123	5,810	114	7,200	119	2,640	109	3,800	107	5,370	105	6,520	107
9	3,060	130	4,400	124	5,830	116	7,070	117	2,600	110	3,810	107	5,470	109	6,420	106
10	3,000	124	4,340	121	5,790	116	7,090	116	2,680	111	4,020	112	5,390	108	6,640	108
11	3,340	140	4,610	127	6,130	120	7,420	125	2,910	122	4,150	114	5,480	108	6,620	111
12	3,260	132	4,560	128	5,910	122	7,290	123	2,530	103	3,920	110	5,290	109	6,460	109
13	3,280	133	4,500	124	5,870	115	7,240	119	2,770	112	3,830	105	5,350	104	6,570	108
14	2,940	123	4,330	119	5,890	117	7,230	120	2,500	105	3,910	107	5,330	106	6,300	104
15	3,000	120	4,370	118	5,610	110	7,020	113	2,550	102	3,940	107	5,120	101	6,430	104
16	3,240	136	4,570	125	6,100	121	7,120	118	2,740	115	4,100	112	5,480	109	6,590	109
17	3,150	128	4,430	125	5,900	115	7,220	119	2,520	102	3,800	107	5,280	103	6,410	106
18	3,270	131	4,500	120	5,860	112	7,050	113	2,880	116	4,050	108	5,420	104	6,470	104
19	3,280	133	4,490	124	5,990	117	7,310	121	2,710	110	3,900	107	5,400	107	6,490	107
20	3,230	138	4,620	131	6,110	122	7,470	123	2,830	121	4,110	117	5,620	113	6,930	115
21	3,200	132	4,440	125	5,830	116	7,340	121	2,780	115	4,090	115	5,550	110	6,750	111
22	2,290	99	3,300	93	4,520	91	5,550	94	1,830	79	2,750	77	3,940	79	4,740	80
23	3,200	139	4,500	129	5,950	122	7,410	123	2,540	108	3,920	109	5,180	105	6,360	105
24	3,470	163	4,580	130	6,030	121	7,490	125	3,140	136	4,010	114	5,280	106	6,470	108
25	2,940	129	4,160	118	5,420	110	6,760	115	2,140	94	3,280	93	4,460	90	5,390	92

[1] Each value is the average of five tests made on five different days. Amount of retarder used is the same amount as shown in table 4.

[2] Ratio represents the ratio (expressed in percent) of the strength of concrete with retarder to the strength of the concrete without retarder made on the same day.

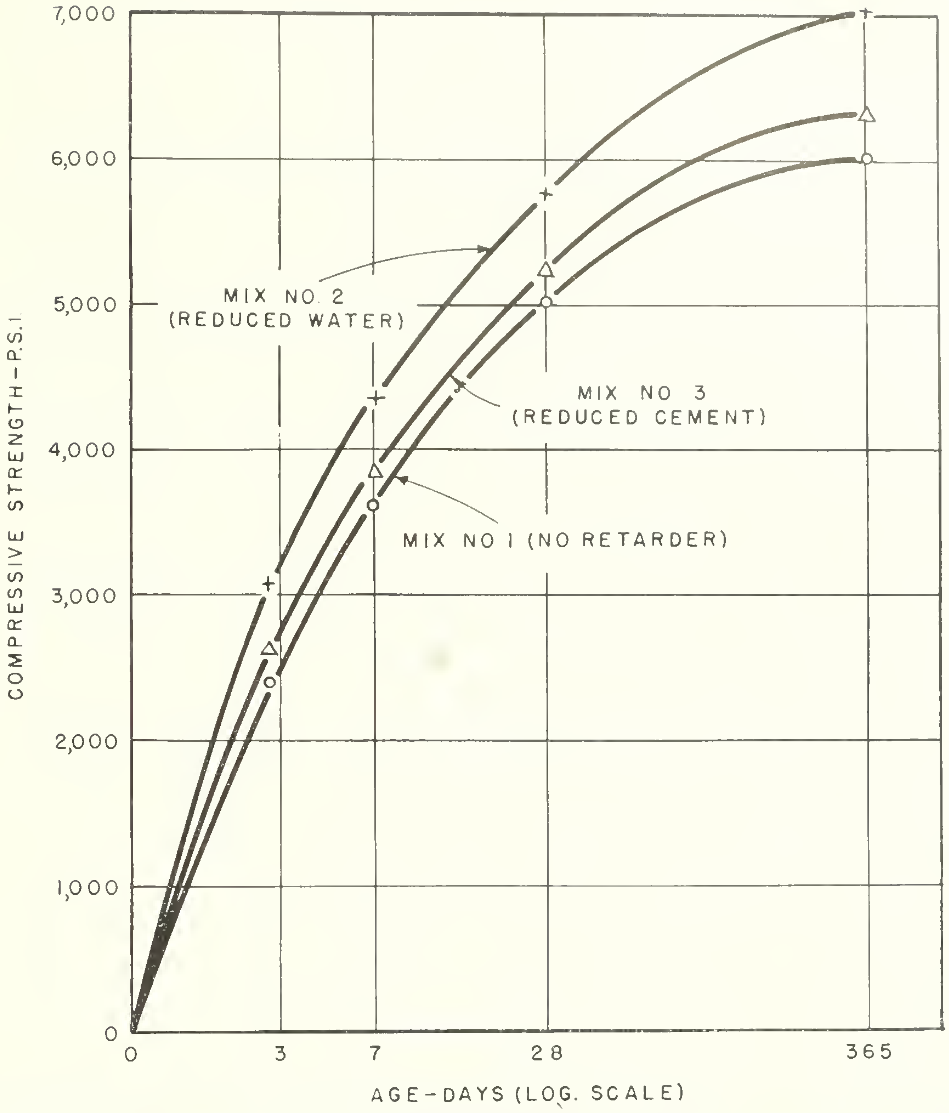


Figure 7.—The influence of retarders on compressive strength of concrete; average values for all retarders.

l- by 16-inch beams which were used for eference purposes in volume change tests. These beams, representing mixes Nos. 1 and 2 nly, had been kept continously in moist torage. When the volume change tests were ompleted, these beams were stored in water or 2 weeks and tested for flexural strength at n age of 15 months. The results obtained re included in table 7 and figure 8.

At an age of 15 months, the flexural strength f nonretarded concrete was the same as that f retarded concrete of reduced water content. t greater ages, the equality in flexural trength of nonretarded concrete and retarded oncrete of reduced water content might be naintained.

Iodulus of elasticity

The results of tests for modulus of elasticity or concretes prepared with each of the 25 etarders are not shown in this report since ignificant differences were not generally btained. However, the average values ecorded for all concretes are of interest. For the reference mix, the sonic modulus ecorded at 7 days was 5.4 million p.s.i., and t 28 days, 5.9 million p.s.i. For mix No. 2, he sonic modulus was 5.8 million p.s.i. at days and 6.2 million p.s.i. at 28 days. For nix No. 3, the sonic modulus was 5.7 million .s.i. at 7 days and 6.1 million at 28 days.

The static modulus was somewhat lower for he three concrete mixes. At 7 days, it was .0 million p.s.i. for the reference mix, 5.4 nillion for mix No. 2, and 5.0 million for mix No. 3. At 28 days, the static moduli for the hree mixes were 5.6 million p.s.i., 6.0 million, nd 5.7 million, respectively.

The sonic moduli for the retarded concretes vere from 3 to 6 percent higher than those for he nonretarded concrete. The static moduli f the retarded concretes were from 1 to 8 percent higher than those for the nonretarded oncretes, but the average static modulus for ll retarded concretes was 7 percent less than he average of the sonic values.

In general, the use of retarders did not ffect adversely the modulus of elasticity of oncrete unless an excessive amount of air was ntrained. Retarder No. 22, which caused ntrainment of an excessive amount of air, roduced concrete having sonic and static noduli from 7 to 13 percent lower than those or nonretarded concrete.

ffect of an elevated temperature

Tests were made to determine the effect of n elevated temperature on the properties of oncrete containing each of the retarders. The materials used were stored for several ays prior to mixing at 90° F. The concrete vas then mixed and the test specimens were ured at the same temperature; the specimens eing cured under wet burlap for 24 hours, and hen stored in moist air at 73° F. until tested.

The amount of retarder necessary to give he desired retardation (2½ to 3 hours as measred by the 500 p.s.i. Proctor test) was deternined and the results are given in table 8. was found that at 90° F. more retarder per ag of cement was required for the concretes ontaining 19 of the retarders than for similar oncretes at 73° F. With the other 6 samples Nos. 14, 15, 16, 17, 20, and 24, all lignosulfonates) the same amount of retarder was required for both temperatures. The increase in amount used for the 19 retarders varied from 105 percent for sample No. 3, to 175 percent for sample No. 6. With respect to type of retarder, the change in temperature from 73° to 90° F. required an increase of 27 percent in the amount of the carbohydrate, and an average increase of 14 percent for lignosulfonates and 48 percent for organic acid retarders.

Retarder No. 5 caused an unusual effect on the setting or stiffening of the screened mortar at 90° F. The mortar developed a firm crust at an early age but remained soft under this crust for some time. When a Proctor penetration pressure of 500 p.s.i. was used, this crust caused the mortar to have an apparent retardation time of only 1 hour and 55 minutes. However, higher penetration pressures of over 500 p.s.i. broke through the crust and revealed the softness of the interior portion of the test specimen. Tests were continued to an age of 9 hours and the penetration pressure was less than 4,000 p.s.i. After 24 hours, the mortar was found to be hard and there was no difficulty in removing the concrete specimens prepared with this retarder from the molds. For a penetration load of 4,000 p.s.i. this retarder retarded the mortar over 4 hours. Tests of the concrete specimens at an age of 3 days indicated a satisfactory strength.

Compressive strength tests for mix No. 2 were made at ages of 3 and 28 days. The results of these tests and the ratios of these strengths to the strength of concrete without retarder, made on the same day, are shown in table 8.

In general, the same trends were shown in tests on specimens made at 90° F. as were shown on those made at 73° F. The strength ratios differed only slightly. The average compressive strength at 3 days of the concretes containing the retarders (mix No. 2) made at 90° F. was 3,020 p.s.i. and for similar concretes made at 73° F. it was 3,080 p.s.i. At 28 days, the average compressive strength of retarded concretes made at 90° F. was 5,040 and for concrete made at 73° F. it was 5,760 p.s.i.

A comparison of the compressive strengths at ages of 3 and 28 days of concrete specimens made at 90° F. and at 73° F. is shown in figure 9. At an age of 3 days, the points representing each concrete, retarded or nonretarded, are well grouped with respect to

Table 7.—Flexural strength tests [1]

Retarder No.	Modulus of rupture [2]									
	Mix No. 2						Mix No. 3			
	7 days [3]		28 days [3]		15 months [4]		7 days [3]		28 days [3]	
	P.s.i.	Ratio [2]	P.s.i.	Ratio [2]	P.s.i.	Ratio [2]	P.s.i.	Ratio [2]	P.s.i.	Ratio [2]
1	665	102	790	110	785	101	645	98	770	108
2	675	112	750	103	780	100	630	105	695	96
3	665	110	750	103	875	105	630	104	745	103
4	725	118	785	104	840	102	620	101	770	102
5	680	106	765	98	870	104	665	104	710	91
6	670	111	755	109	830	99	590	98	710	102
7	675	107	760	100	845	101	645	102	730	96
8	690	108	830	112	885	106	630	98	755	102
9	690	109	805	106	855	104	655	103	745	98
10	670	104	805	113	740	90	690	107	750	106
11	665	108	725	95	835	102	650	106	710	93
12	720	118	760	105	760	97	620	102	685	94
13	685	107	785	102	805	96	655	102	725	94
14	675	109	800	102	850	104	625	101	750	96
15	715	111	770	110	845	101	645	100	685	98
16	630	102	730	94	890	109	610	98	705	90
17	690	112	750	103	815	98	655	106	745	103
18	660	107	755	97	810	99	600	98	685	88
19	720	110	790	103	755	90	675	103	755	98
20	695	107	800	109	815	99	630	97	730	99
21	660	104	800	109	825	101	640	101	740	101
22	595	98	695	96	675	87	535	88	640	89
23	670	108	810	114	715	92	625	100	750	107
24	635	106	745	101	810	104	600	100	650	88
25	655	107	750	105	775	99	590	97	660	92

[1] Amount of retarder used is the same amount as shown in table 4.

[2] Ratio represents the ratio (expressed in percent) of the strength of concrete with retarder to the strength of the concrete without retarder made on the same day.

[3] Each value is the average of five tests made on 5 different days. Specimens were 6- x 6- x 21-inch beams tested in accordance with AASHO Method T 97 with third point loading on an 18-inch span. Side as molded in tension.

[4] Each value is the average of three tests. Specimens were 3- x 4- x 16-inch beams tested with third point loading on a 12-inch span with a 4-inch depth; bottom surface as molded in tension.

the 45° line. The concretes made with retarders Nos. 3 and 25 show the greatest reduction in strength at 3 days due to fabrication at an elevated temperature. At an age of 28 days, almost all of the concretes made at 90° F. show a lower strength. Concretes prepared with retarders Nos. 5, 7, 11, 13, 18, and 25 show this to the most marked extent. With respect to type of retarder, some differences are found but no outstanding influence of any one type is noted. Concretes prepared with retarders Nos. 3, 6, and 15, showed either none or only slight reduction in strength due to the elevated temperature.

Freezing and thawing tests

Table 9 shows the results of freezing and thawing tests made with the retarded concrete. The computed durability factors for concrete prepared with each retarder after 300 cycles of freezing and thawing are given in this table. Because of the limited capacity of the freezing and thawing apparatus the specimens for these tests were prepared on 10 mixing days.

To permit comparisons between the concretes containing the different retarders a reference mix was made on each mixing day. The specimens made with the reference mix were tested with the specimens containing the retarders, and the durability factors for each concrete of mixes Nos. 2 and 3, expressed as a percentage of the durability factor for the reference concrete made on the same day, are shown in table 9. These values are described as relative durability factors. The durability factors for the reference concrete were quite uniform, varying from 85 to 92 with an average of 90.

Concretes of equal cement content (mix No. 2) prepared with each of 3 retarders, Nos. 12, 15, and 24, had relative durabilities of 80 percent or less whereas concretes with the remaining 22 retarders had relative durabilities of 90 percent or more. Concretes of reduced cement content (mix No. 3) prepare with the same 3 retarders had relative durabilities of less than 90 percent. Concrete containing retarders Nos. 3 and 23 also ha relative durabilities of less than 90 percen when the cement content was reduced.

If an assumption is made that a relativ durability of 80 percent for concrete with a admixture is acceptable, as is specified i AASHO Specification M 154 for air-entrainin admixtures for concrete, and in the propose specification for retarders of Subcommitte III–h of ASTM Committee C–9, then only tw retarders, Nos. 15 and 24 for mix No. 2, an three retarders, Nos. 3, 15 and 24 for mix No. would not be acceptable.

Retarders Nos. 3, 15, and 24 were all of th lignosulfonate type. The air contents of th concretes containing these retarders varie from 5 to 6.5 percent and with one exceptio (sample No. 24 in mix No. 3) no air-entrainin agent was added. Concretes containing fo other lignosulfonate retarders, Nos. 20, 22, 2 and 25, to which no air-entraining agent wa added to six of the eight mixes, had relativ durability factors for mixes Nos. 2 and 3 var ing from 86 to 103 percent. Concretes co taining the other lignosulfonates to which a air-entraining agent was added, with the e ception of retarder No. 12 in both mixes No 2 and 3, had durability factors of over 9 percent.

Organic acid retarders Nos. 4, 5, 6, 8 and which required considerable Vinsol resin solution to entrain the required amount of air i concretes for mixes Nos. 2 and 3, had relativ durability factors in excess of 90 percen Considerable amounts of air-entraining agen were also required for mixes No. 2 and 3 wit

Table 8.—Retardation time for concrete and compressive strength tests of specimens mad at 90° F. for mix No. 2

Retarder No.	Amount retarder used, 73° F.	Amount retarder used, 90° F.	Slump	Air	Proctor penetration test, retardation at—[1]		Compressive strength			
							3 days		28 days	
					500 p.s.i.	4,000 p.s.i.	P.s.i.	Ratio [2]	P.s.i.	Ratio [2]
	Oz./bag	*Oz./bag*	*Inches*	*Percent*	*Hr. Min.*	*Hr. Min.*				
1	1.1	1.4	3.2	4.9	3:05	2:50	2,880	121	5,090	115
2	14.4	16.3	3.2	5.8	2:45	2:10	2,550	107	4,410	99
3	4.0	4.2	3.1	5.6	2:35	2:35	3,250	132	5,260	125
4	3.0	4.2	4.4	5.5	2:20	2:15	2,890	117	5,220	119
5	4.0	6.0	4.4	5.8	[3] 1:55	4:+	3,220	135	5,080	114
6	4.0	7.0	3.6	5.8	2:45	3:35	2,770	116	5,400	122
7	3.5	4.5	3.0	5.4	2:40	2:55	2,880	117	4,700	112
8	12.0	16.0	3.3	5.1	2:35	2:50	3,210	130	5,330	121
9	2.8	4.0	2.9	6.0	3:10	3:20	3,060	124	5,220	119
10	3.2	4.3	3.2	5.2	3:10	3:00	3,090	129	5,280	119
11	4.8	6.0	2.9	5.8	2:50	2:35	3,470	140	5,070	115
12	4.0	5.3	2.7	6.9	2:35	2:55	2,840	119	5,070	114
13	4.8	5.3	2.8	5.3	3:05	2:50	2,810	114	4,540	108
14	5.0	5.0	2.7	5.6	2:20	2:30	3,180	129	5,210	119
15	4.0	4.0	3.0	4.8	2:15	2:25	3,320	134	5,470	125
16	4.8	4.8	3.0	5.3	2:20	2:00	3,350	136	5,350	122
17	3.2	3.2	2.9	5.2	3:00	2:40	2,960	120	5,270	120
18	4.0	5.0	2.9	6.5	2:25	2:40	3,060	124	4,650	111
19	4.0	4.5	2.8	5.0	2:40	2:15	3,160	128	5,390	128
20	9.0	9.0	2.9	5.4	2:30	2:25	3,030	123	5,170	123
21	9.0	12.0	2.7	5.3	2:40	2:55	3,110	130	5,420	122
22	4.0	4.5	3.5	8.+	2:25	2:45	2,340	95	3,940	94
23	6.4	7.2	3.0	5.0	2:30	2:25	3,020	126	5,100	115
24	9.6	9.6	3.0	5.4	2:40	2:30	3,580	146	5,180	123
25	4.0	5.0	2.7	7.1	2:25	2:02	2,410	101	4,280	96

[1] The delay in time of hardening of the concrete containing the retarder as compared with the concrete without retard made on the same day. Time for concrete without retarder to reach: Proctor penetration load of 500 p.s.i.: 3 hrs. 15 mi (±15 min.); Proctor penetration load of 4,000 p.s.i.: 4 hours 45 min. (±15 min.).

[2] Ratio represents the ratio (expressed in percent) of the strength of concrete with retarder to the strength of concrete wit out retarder made on the same day.

[3] Top surface appeared to crust.

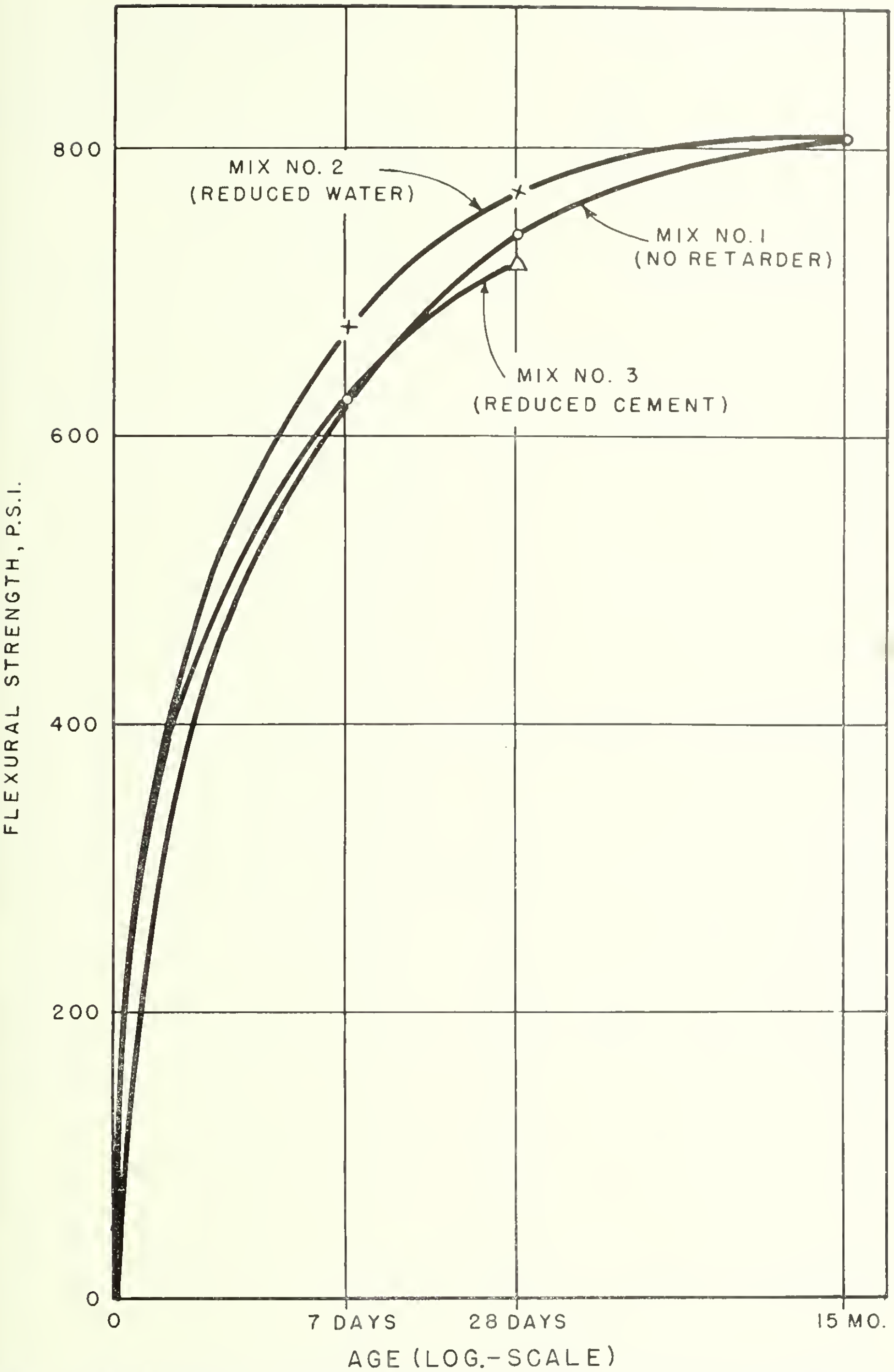


Figure 8.—The influence of retarders on flexural strength of concrete; average values for all retarders.

sample No. 1, the carbohydrate retarder. The relative durability for this concrete was 101 percent for mix No. 2 and 99 percent for mix No. 3.

Concrete having an air content of 5 to 7 percent is usually considered to have an adequate resistance to the effects of freezing and thawing. The fact that the concretes prepared with retarders Nos. 3, 15, and 24 contained at least 5 percent of air and still showed relatively poor resistance to freezing raises a question of the beneficial value of air entrained by some lignosulfonates. The only explanation which might be given to this is that the air was not in the finely divided and uniformly dispersed form most advantageous in furnishing durability to concrete. However, no study of air void size and spacing was made. The information obtained in these tests indicated that freezing and thawing tests may be of particular significance in acceptance or qualification tests of a retarder which requires no air-entraining admixture to obtain the specified air content in concrete.

Effects of delayed vibration

The results of tests to determine the effects of delayed vibration on the properties of concretes prepared with retarders are shown in table 10. During three rounds of making specimens for flexure and compression tests, an extra beam and cylinder were made for each batch. The three extra beams and three cylinders were vibrated at 3 or 5½ hours after molding. The beams and cylinders prepared with mix No. 1 were vibrated at 3 hours and those prepared with mixes Nos. 2 and 3 were vibrated at 5½ hours. The unit weight of each concrete, vibrated and nonvibrated, was determined by weighing cylinders in air and under water prior to testing at an age of 28 days.

The average increase in unit weight of all vibrated retarded concretes was 3.1 and 3.4 pounds per cubic foot for mixes Nos. 2 and 3, respectively, over the unit weights of corresponding concretes from the same batch which had not been vibrated. In general, a greater increase in weight was obtained by vibration of the concretes which had high air contents. For example, concretes of an air content of 8 or more percent (retarder No. 22) showed increases in unit weight of 6.5 and 7.4 pounds after vibration.

The average unit weights of the vibrated, reference mix prepared on 18 mixing days was 151.3 pounds per cubic foot and 148.3 pounds for the corresponding reference mix that was not vibrated. The average gain in weight of 3 pounds per cubic foot for the nonretarded mix checks with the average increase in weight of 3.1 pounds for mix No. 2 and 3.4 pounds for mix No. 3.

The average compressive strength at an age of 28 days of vibrated, nonretarded concrete (mix No. 1) was 5,680 p.s.i. This value was 11 percent higher than that for the same mix which was not vibrated. Vibration did not improve the retarded concretes to the same amount. The 28-day compressive strengths of vibrated retarded concrete were 4.1 and 7.3 percent higher, for mixes Nos. 2 and 3, respectively, than the strengths of the same concrete which was not subjected to delayed vibration. All vibrated concrete for mix No. 3 and 22 of the 25 samples tested in mix No. 2 showed a gain in strength over nonvibrated concrete.

The 28-day flexural strength of vibrated mix No. 2 averaged 2.9 percent lower than the strength of the same mix without delayed vibration. For mix No. 3 the vibrated and nonvibrated concretes had about the same flexural strength.

The average flexural strength of the vibrated, nonretarded concretes (mix No. 1) was 725 pounds per square inch. This was about one percent less than the average strength of the corresponding nonvibrated concrete.

The tests indicated that delayed vibration of retarded concrete at 5½ hours after placing was beneficial to the compressive strength. However, the time required for the 500-p.s.i. Proctor penetration pressure for delayed vibration was not applicable to concretes which were used in this study. This was probably due to the grading and the type of aggregate used.

The results of delayed vibration on flexural strength of retarded concrete included in this investigation indicate no apparent benefit, probably due to the method of vibration

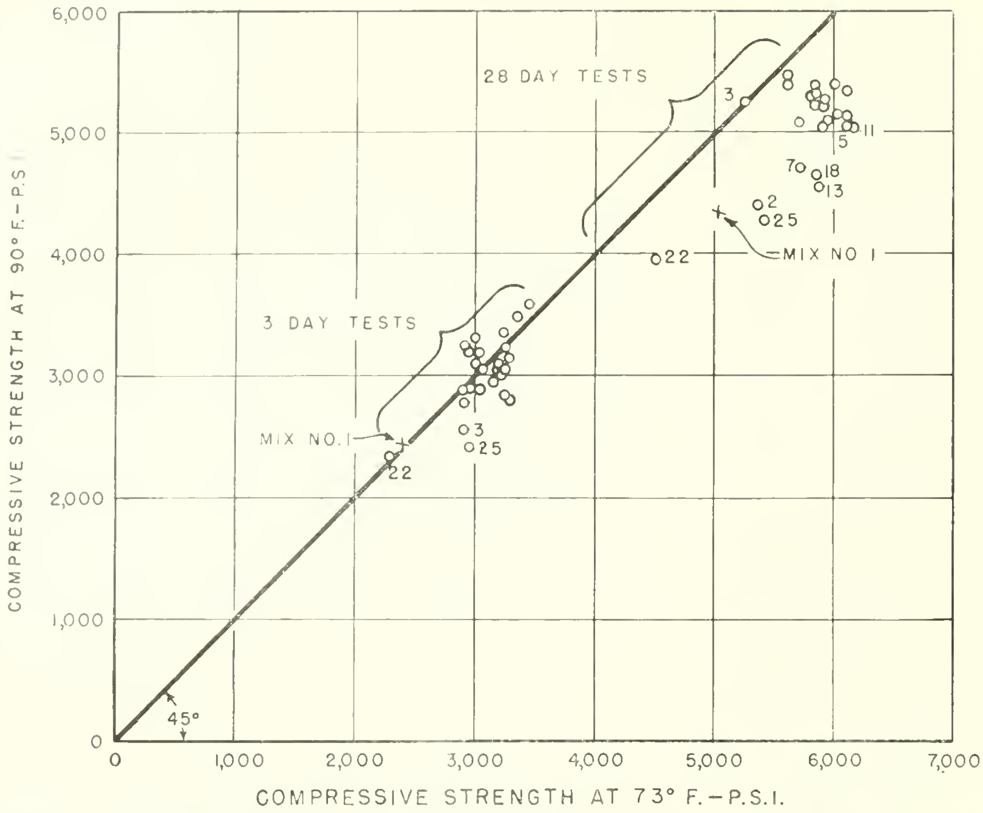


Figure 9.—Effect of elevated temperature on compressive strength of retarded concrete.

employed. It is difficult to vibrate a horizontal beam of 6- by 6-inch cross-section with a 1⅛-inch internal vibrator and obtain ideal compaction. In the case of a 6- by 12-inch cylinder, the vibrator spud is immersed for its full length in the concrete and effective vibration can be obtained.

Density and absorption tests

Data on the density and absorption of concrete disks taken from cylinders prepared with and without retarders are shown in table 11. Mix No. 2, containing each of the 25 retarders, was made on three different mixing days along with a reference mix prepared for each day. The tests were made on 1-inch thick disks which were sawed from the top, middle and bottom of three 6- by 12-inch cylinders made on the same day. Each value for weight or absorption of the hardened concrete given in the table is an average for 6 disks.

A temperature of 190° to 200° F. was used to dry the disks. This temperature was chosen because it was considered inadvisable to evaporate water above the boiling point and the uniformity of heating was better controlled at this temperature than in available ovens which operated at temperatures about 212° F.

The wet and dry densities, unit weight per cubic foot, of the disks from the bottom of the cylinders were higher than the densities of corresponding disks from the top of the cylinders. However, the differences in densities of disks from the top and middle of cylinders were relatively small and indicated very little effect on densities by any of the retarders. A comparison of densities of the retarded concretes with the densities of the reference mix also indicates a non-appreciable gain in weight above that normally obtained for the reduced water content, and the term "densifier" as used for some retarders, is not justified by these data.

The absorption data given in table 11 were calculated from the wet weights at an age of 28 days and the weights after drying at 190° to 200° F. The absorptions were higher for the disks from the top of the cylinders than for the disks from the bottom of the cylinders. There did not appear to be any appreciable effect o decreasing the absorption of the concrete in th top of a cylinder by the use of a retarder Neither did the use of a retarder appear t decrease appreciably the range in absorptio between the concrete in the top and bottom o a cylinder.

Volume change tests

The data in table 12 indicate that retarder have no significant effect on lineal drying shrinkage or growth of the concrete used i this study. This conclusion is based on comparison of the volume change of retarde concrete with that of reference concrete whic was made on the same day. In general, th organic acid retarders (Nos. 4, 5, 6, 8, and 9 and the carbohydrate retarder (No. 1 showed very little increase in shrinkage. Th shrinkage values, generally, appear to b higher for the retarded concretes that wer moist cured initially for 2 days than those fo similar concretes that were cured initially fo 14 days. Some of the lignosulfonates, pa ticularly Nos. 11, 14, 16, 18, and 24, showe the greatest shrinkage for 2-day moist-cure concrete. Whether the calcium chloride co tent of retarders Nos. 11 and 24 affecte shrinkage is subject to conjecture. Likewis whether reducing sugar content, over fiv percent in retarders Nos. 14 and 17, affecte shrinkage is conjectural. Although the shrink age values for concretes containing retarde Nos. 14 and 17 were among the highest valu recorded, consideration should be given th chemical composition of each of the tw retarders.

The values for expansion under mois storage showed no appreciable effect of r tarders on concrete. One value, 0.019 pe cent, for retarder No. 14 is an exception an reference to table 1 shows that the carbohy

Table 9.—Results of freezing and thawing tests [1]

Retarder No.	Mix No. 2				Mix No. 3			
	Air content	AEA added	Durability factor at 300 cycles	Relative durability factor [2]	Air content	AEA added	Durability factor at 300 cycles	Relative durability factor [2]
	Percent	*Ml./bag*	*Percent*	*Percent*	*Percent*	*Ml./bag*	*Percent*	*Percent*
1	4.9	16.3	92	101	5.4	21.5	91	99
2	5.2	16.3	91	100	5.6	21.5	91	99
3	6.5	------	87	96	5.6	------	68	75
4	5.2	14.3	91	101	5.1	18.0	89	99
5	5.1	10.2	87	96	4.9	13.5	82	91
6	5.3	14.3	91	100	5.4	20.2	85	94
7	6.1	10.2	90	99	5.5	13.5	87	96
8	5.0	15.1	91	99	5.3	19.3	91	106
9	4.9	14.3	94	102	5.3	18.0	87	97
10	6.0	18.4	91	99	5.7	13.5	88	97
11	5.4	4.1	88	95	5.3	9.0	84	93
12	5.7	4.1	74	80	5.9	6.7	72	85
13	5.3	4.1	88	96	6.0	6.7	79	93
14	6.0	4.1	87	94	5.3	11.2	84	99
15	6.3	------	72	79	5.0	------	64	75
16	6.2	6.1	92	101	5.5	6.7	83	98
17	5.2	10.2	92	102	5.2	13.5	86	101
18	6.2	6.1	90	99	5.0	6.7	82	93
19	5.9	8.2	91	101	5.0	11.2	87	98
20	5.1	------	85	94	5.9	13.5	91	103
21	5.0	4.1	82	91	5.2	11.2	92	103
22	10.0	------	92	101	7.9	------	91	102
23	6.1	4.1	88	97	5.0	4.7	76	86
24	5.0	------	56	61	5.9	2.4	68	76
25	7.6	------	87	96	6.6	------	87	97

[1] Each value is an average of tests on three 3- x 4- x 16-inch beams. Beams frozen and thawed in accordance wi AASHO Method T 161 for fast freezing and thawing in water.

[2] Relative durability factor is the ratio in percent of the durability factor of the concrete containing the retarder to t durability factor of concrete without retarder (mix No. 1) made on the same day and given the same treatment.

drate content for this retarder is over 10 percent.

Table 10.—Effect of delayed vibration on compressive and flexural strength of concrete at 28 days[1]

Retarder No.	Weight of hardened concrete[2]		Compressive strength, vibrated		Flexural strength, vibrated	
	Vibrated	Non-Vibrated	P.s.i.	Ratio[3]	P.s.i.	Ratio[3]
	Pounds per cu. ft.	*Pounds per cu. ft.*				
Mix No. 1	151.3	148.3	5,680	111	725	99
Mix No. 2:						
1	151.8	149.8	6,070	105	765	96
2	151.7	148.5	5,680	101	740	97
3	151.8	147.8	5,790	113	665	92
4	151.9	148.8	5,930	100	775	97
5	153.5	150.3	6,500	101	755	97
6	151.2	147.6	5,550	104	715	95
7	152.9	150.8	5,950	103	750	101
8	152.5	150.1	6,050	101	790	94
9	151.2	149.7	6,240	107	725	88
10	152.4	150.2	6,120	105	765	96
11	152.2	147.0	6,240	102	735	105
12	150.8	145.3	5,690	116	740	106
13	152.6	150.7	6,140	104	760	101
14	153.8	150.9	6,270	105	775	97
15	152.3	149.1	5,880	103	725	97
16	152.4	150.0	6,170	100	765	103
17	152.3	150.1	5,900	98	740	97
18	152.8	151.4	5,740	94	760	101
19	153.8	151.1	6,170	102	750	93
20	152.5	150.6	6,260	102	790	100
21	152.1	149.9	6,210	105	745	94
22	150.8	144.3	5,420	125	705	104
23	152.3	149.8	6,290	102	790	95
24	154.0	151.1	6,020	98	725	93
25	153.3	148.1	6,120	109	745	95
Average	152.4	149.3	------	104	----	97
Mix No. 3:						
1	151.6	147.6	5,810	106	695	91
2	152.0	148.4	5,360	104	730	105
3	152.2	148.6	5,610	112	710	97
4	152.0	148.7	5,560	105	725	94
5	153.5	149.8	6,270	109	705	96
6	151.1	148.9	5,320	106	675	97
7	152.2	149.8	5,830	105	735	104
8	152.4	149.7	5,690	103	725	96
9	152.4	149.3	5,910	105	755	102
10	153.2	150.7	6,000	108	720	99
11	153.6	149.7	5,710	103	685	97
12	151.7	147.8	5,360	107	745	106
13	153.8	149.6	5,890	109	720	104
14	153.0	149.3	5,610	105	740	100
15	153.5	148.8	5,950	114	670	97
16	153.2	150.0	5,690	101	735	101
17	152.1	149.4	5,790	109	750	101
18	152.6	150.5	5,600	100	760	113
19	152.8	150.1	5,790	105	735	93
20	154.0	151.0	6,380	108	680	95
21	152.5	150.5	5,800	103	720	104
22	152.4	145.0	5,190	126	670	105
23	153.0	149.9	5,580	104	765	103
24	153.6	150.7	5,600	105	670	105
25	152.6	147.6	5,880	126	670	98
Average	152.7	149.3	--------	107	--------	100

[1] Each value for mix No. 1 is average of 19 tests. Each value for mixes Nos. 2 and 3 is average of 3 tests made on different days.

[2] Weight of hardened concrete determined on test cylinders at age of 28 days.

[3] Ratio represents the ratio (expressed in percent) of the strength of vibrated concrete to the strength of the non-vibrated concrete made on the same day.

Use of excessive amount of retarder

A limited series of tests were made using four times the amount of retarder which had previously been determined as the amount necessary to obtain the desired retardation. Only ten of the 25 retarders were used in these tests which were made to determine the effect on the retardation as well as on compressive strength at ages of 7 and 28 days. The mix data for these concrete samples as well as the results of the Proctor penetration tests and the compressive strength tests and ratios, are given in table 13.

The mix data show that when four times the normal amount of retarder was used, the air content of the concrete mixtures Nos. 3, 7, and 11 was over 8 percent. No air entraining admixture was used in these mixtures. The use of an overdosage of retarder had no effect on the air content of the concretes containing each of the other seven retarders. The water required for the concretes with an overdosage of each retarder was, with one exception, reduced from that required for corresponding concretes containing the normal amount of retarder.

The time required for the concretes to reach 500 p.s.i. penetration pressure by the Proctor test, varied from approximately 6 hours for concrete containing retarder No. 11 to over 100 hours for retarders Nos. 3 and 7. For a penetration of 4,000 p.s.i., the time varied from 23 hours to over 150 hours. Retarders Nos. 11 and 12 which required approximately to 10 hours, respectively, for a 500 p.s.i. penetration pressure, required 80 hours and over 150 hours, respectively, for 4,000 p.s.i. penetration pressure. Crusting of the top surface of the concrete appears to have caused his difference.

The strength tests show that at an age of 7 days, concrete prepared with retarders Nos. 1, 2, and 9 gave compressive strengths greater han those of the concrete without retarder. Concrete containing these 3 retarders reached he Proctor penetration value of 4,000 p.s.i. n less than 50 hours.

At an age of 28 days the concretes containing each of six retarders had strengths greater than the strength of the nonretarded nix. Of the four concretes which had trengths less than the reference concrete, hree had air contents of over 8 percent. The concretes containing retarders Nos. 7 and 12 ad strength ratios of 90–91 percent of the onretarded concrete. As these concretes eeded over 100 hours to obtain a Proctor penetration pressure of 4,000 p.s.i., it is possible that at a later age these concretes might equal the strength of the reference mix.

Where the air content of the concrete was ot high, it appeared that if sufficient curing could have been given the concrete, the strength would not have been seriously owered by the overdosage of retarder.

Figure 10 shows the probable effect of retarder overdosage on the compressive strength of concrete after 7 and 28 days. The curves showing the effect of retarders Nos. 1, 2, and 9 have approximately the same shape as that for the reference mix. The curves for the concrete retarders Nos. 3 and 11 have about the same slope as that for the reference mix. It was expected that these concretes would continue to develop strength at about the same rate as the reference concrete, and that little change in the retarded and nonretarded strength ratios would occur with increase in age.

The curves for the concretes prepared with retarders Nos. 4, 5, 6, 7, and 12 show a very rapid increase in strength after a prolonged delay in hardening. The assumption drawn from the plotted data was that for these retarders a considerable delay in setting caused a very rapid gain in strength, and possibly a development of strength superior to that when normal rates of hardening occurred. This assumption, however, does not apply to all retarders which had a marked delay in setting as samples Nos. 3 and 11 had such delay but failed to gain a normal amount of strength between ages of 7 and 28 days. For most of the retarders tested, use of an excessive amount did not cause an objectionable reduction in strength at an age of 28 days.

Effect of retarders on temperature rise

Figure 11 shows the results of temperature measurements of retarded and nonretarded concretes which were stored at 73° F. for 48 hours. After mixing, retarded concretes required a longer period of time to reach the maximum temperature than did the reference mix. The increase varied from approximately

Table 11.—Density and absorption tests for mixes Nos. 1 and 2

Retarder No.	Weight plastic concrete (pounds per cubic foot)	Air in plastic concrete (percent)	Weight of hardened concrete (pounds per cubic foot)						Absorption [3] (percent)		
			Wet [1]			Dry [2]					
			Top	Middle	Bottom	Top	Middle	Bottom	Top	Middle	Bottom
Mix No. 1	145.7	4.5	150.8	151.6	152.0	142.6	143.6	144.1	5.8	5.6	5.5
4	145.9	4.9	151.0	150.8	152.6	143.1	142.8	145.2	5.5	5.6	5.1
9	145.3	5.1	149.5	150.2	152.5	141.2	141.9	144.8	5.9	5.8	5.3
10	144.3	5.5	119.8	150.1	152.6	141.5	142.1	145.3	5.9	5.6	5.0
11	145.9	4.7	151.9	152.6	154.2	144.1	144.6	147.0	5.4	5.5	4.9
14	145.9	4.9	152.1	152.1	153.3	144.4	144.3	146.0	5.3	5.4	5.0
16	144.3	6.5	151.0	151.0	152.7	143.6	143.3	145.8	5.2	5.4	4.7
18	145.7	5.0	151.2	151.4	154.1	143.1	143.3	146.9	5.7	5.7	4.9
20	144.3	6.0	149.5	150.1	152.4	141.2	142.2	145.0	5.9	5.6	5.1
21	144.7	5.6	150.9	151.2	152.8	143.1	143.2	145.5	5.5	5.6	5.0
Mix No. 1	144.3	5.0	149.6	149.8	151.7	141.7	141.7	144.5	5.6	5.7	5.5
3	143.1	6.5	151.6	150.3	151.8	144.3	142.6	144.7	5.1	5.4	4.9
5	145.5	5.1	152.6	151.6	153.3	145.5	144.1	146.5	4.9	5.2	4.6
6	144.7	5.2	150.1	149.7	152.0	142.3	141.8	144.8	5.5	5.6	5.0
7	144.9	5.3	151.2	150.5	152.5	143.7	142.9	145.7	5.2	5.3	4.7
8	144.1	5.5	150.3	150.1	152.0	142.5	142.3	144.9	5.5	5.5	4.9
13	143.1	6.4	149.2	149.3	151.2	141.5	141.7	144.3	5.4	5.4	4.8
15	141.9	6.7	148.8	149.0	150.9	141.1	141.4	144.2	5.5	5.4	4.6
17	144.5	5.9	151.0	150.2	152.4	143.5	142.5	145.6	5.2	5.4	4.7
19	145.5	5.0	150.9	150.6	152.5	142.9	142.6	145.3	5.6	5.6	5.0
Mix No. 1	143.5	5.0	148.3	149.1	150.6	139.5	140.5	142.7	6.3	6.1	5.5
1	144.3	5.2	149.0	149.5	151.8	140.5	141.3	144.6	6.0	5.8	5.0
2	143.9	5.2	148.2	148.4	151.1	139.7	139.9	143.4	6.1	6.1	5.4
12	144.3	5.1	151.9	151.7	152.8	141.4	144.2	145.7	5.2	5.2	4.9
22	138.6	8+	144.7	144.7	147.3	136.8	136.9	140.3	5.8	5.7	5.0
23	144.9	5.1	150.1	149.6	152.4	142.1	141.5	145.2	5.6	5.7	5.0
24	141.5	6.0	150.2	150.1	151.6	142.6	142.7	144.8	5.3	5.2	4.7
25	140.8	8+	147.8	147.9	150.0	139.8	140.0	141.9	5.7	5.6	5.7

[1] Wet weight determined on disks after 28 days continuous moist storage.
[2] Disks dried to constant weight at 190 to 200° F.
[3] Based on wet weight after 28 days moist curing.

Table 12.—Tests for change in volume fo mixes Nos. 1 and 2 [1]

Retarder No.	Contraction [2]		Expansion [3]
	Moist cured 2 days	Moist cured 14 days	
	Percent	*Percent*	*Percent*
Mix No. 1	0.044	0.048	0.009
4	.046	.041	.011
9	.046	.044	.011
10	.047	.047	.013
11	.059	.048	.012
14	.052	.048	.019
16	.055	.049	.012
18	.057	.049	.008
20	.049	.046	.011
21	.047	.048	.011
Mix No. 1	.049	.046	.009
3	.049	.045	.011
5	.052	.044	.010
6	.044	.046	.011
7	.051	.047	.011
8	.044	.040	.010
13	.053	.045	.010
15	.047	.046	.012
17	.053	.056	.011
19	.052	.046	.010
Mix No. 1	.048	.044	.008
1	.046	.047	.011
2	.048	.048	.008
12	.047	.039	.009
22	.049	.044	.014
23	.052	.044	.009
24	.056	.051	.011
25	.053	.042	.011

[1] Each value is an average for three 3- x 4- x 16-inch beams
[2] Specimens stored in dry air for 200 days at 73° F. and 5 percent relative humidity.
[3] Specimens stored in moist air for 200 days at 73° F. an 100 percent relative humidity.

one to seven hours. In only three samples were the maximum temperatures of retarded concretes higher than that of the reference mix. The greatest maximum temperature, 101° F., was found with retarder No. 24. This temperature was reached almost 7 hours after the maximum temperature of the reference concrete had been reached. The lowest maximum temperature was 90° F., obtained with retarder No. 5.

The results of measurements for temperature rise of retarded and nonretarded mortars which were screened from concrete made at 90° F. are shown in figure 12. The mortar specimens were stored in air at 90° F. for 24 hours. The increase in time for retarded mortars to reach a maximum temperature over the time required for nonretarded mortars varied from about 2½ to 16 hours. The greatest maximum temperature, 120° F., was obtained with retarder No. 24 and the lowest maximum temperature, 105° F., occurred with retarder No. 6. Somewhat of an oddity occurred with mortar containing retarder No. 5. This sample recorded the second lowest maximum temperature, 106° F., at two different times. The first temperature maximum was reached after five hours—which was prior to the time the nonretarded mortar had reached its maximum temperature—and retarder No. 5 reached 106° F. again after 23 hours.

The data show, with the single exception of No. 5 on the tests with mortar specimens, that all retarders delay the time of maximum temperature rise beyond that of nonretarded concrete and mortar. The data also show that, in general, retarders do not increase appreciably the maximum temperature rise but may decrease it. Retarder No. 24, however, presents an exception to the last statement as this sample caused both mortar and concrete to develop a significantly highe temperature than the nonretarded mixes.

Discussion of the Data

It is to be noted that the findings of thi investigation apply specifically to the concret prepared with the following materials an properties:

Cement.—Blend of four type I cements having a calculated average equivalent alkali content of 0.63 percent.

Aggregates.—Natural quartz sand, and crushed limestone of 1-inch maximum size.

Slump.—2 to 3 inches.

Air content.—5 to 6 percent.

Mixing.—In laboratory pan-type mixer.

Table 13.—Tests using four times normal amount of retarder for mixes Nos. 1 and 2 [1]

Retarder No.	Amount retarder used	AEA added	Cement	Water	Slump	Air	Proctor penetration test time for—		Compressive strength at [2]—			
							500 p.s.i.	4000 p.s.i.	7 days		28 days	
									P.s.i.	Ratio [2]	P.s.i.	Ratio [2]
	Oz./bag	*Ml./bag*	*Bags/cu. yd.*	*Gal./bag*	*Inches*	*Pct.*	*Hr. Min.*	*Hr. Min.*				
Mix No. 1	-----	27	6.0	5.9	2.7	5.4	3:50	5:25	3,650	--------	4,900	--------
1	4.5	23	6.0	5.7	2.8	5.1	20	23	4,330	119	5,500	112
2	57	17	5.9	5.8	2.9	5.1	27	29	4,160	114	5,560	113
3	16	---	5.2	5.3	5.4	8+	100+	100+	1,180	41	2,640	54
4	12	25	6.0	5.4	3.0	5.1	80+	80+	2,380	65	5,900	120
5	16	12	6.0	5.3	4.3	4.8	15±	75	3,340	92	6,200	127
6	16	20	5.9	5.4	3.2	5.7	80+	150+	320	9	5,030	103
7	14	-----	5.8	5.2	5.2	8+	100+	150+	--------	--------	4,420	90
9	11	25	5.9	5.4	3.8	5.6	40±	48	3,860	106	5,650	115
11	19	--	5.2	4.9	4.2	8+	6	80+	1,540	42	2,500	51
12	16	27	6.0	5.1	2.5	5.5	10±	150+	--------	--------	4,460	91

[1] Normal amount of retarder is amount required to retard the setting time of the concrete at 73° F. for 2½ hours to 3 hours.

[2] Ratio represents the ratio (expressed in percent) of the strength of concrete with retarde to the strength of the concrete without retarder.

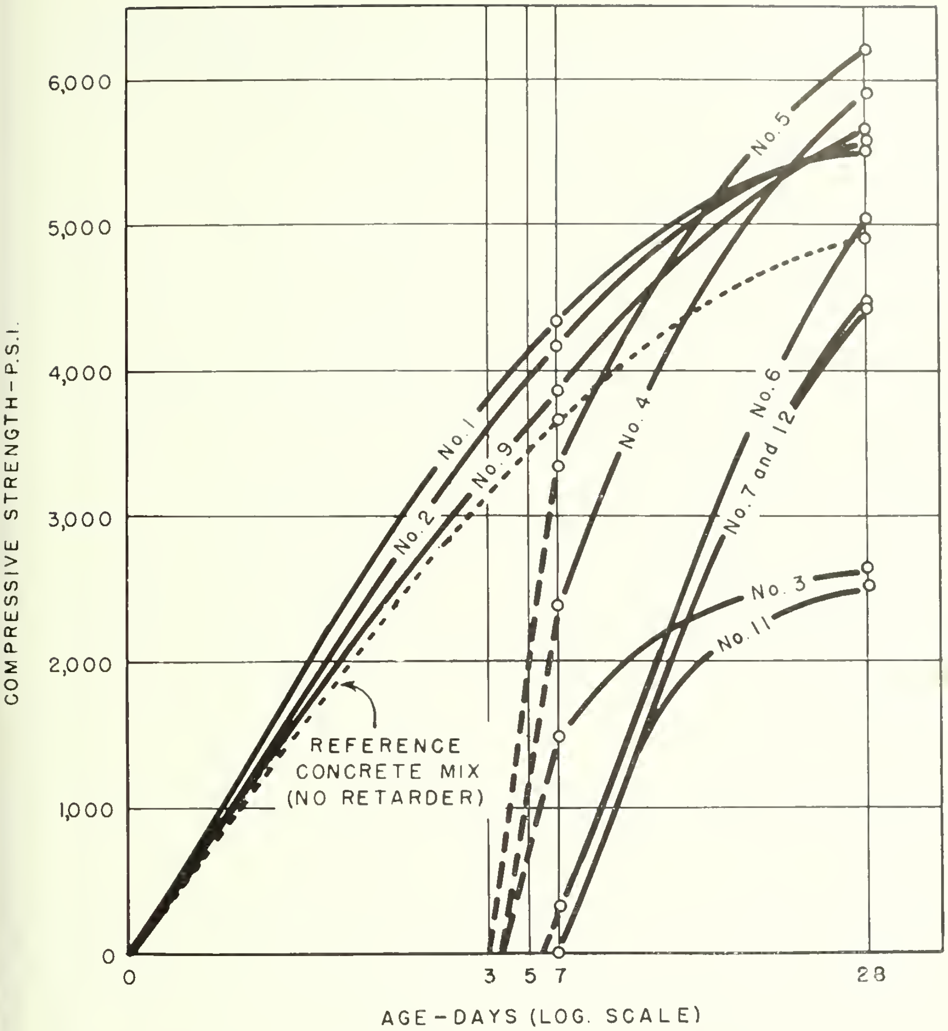
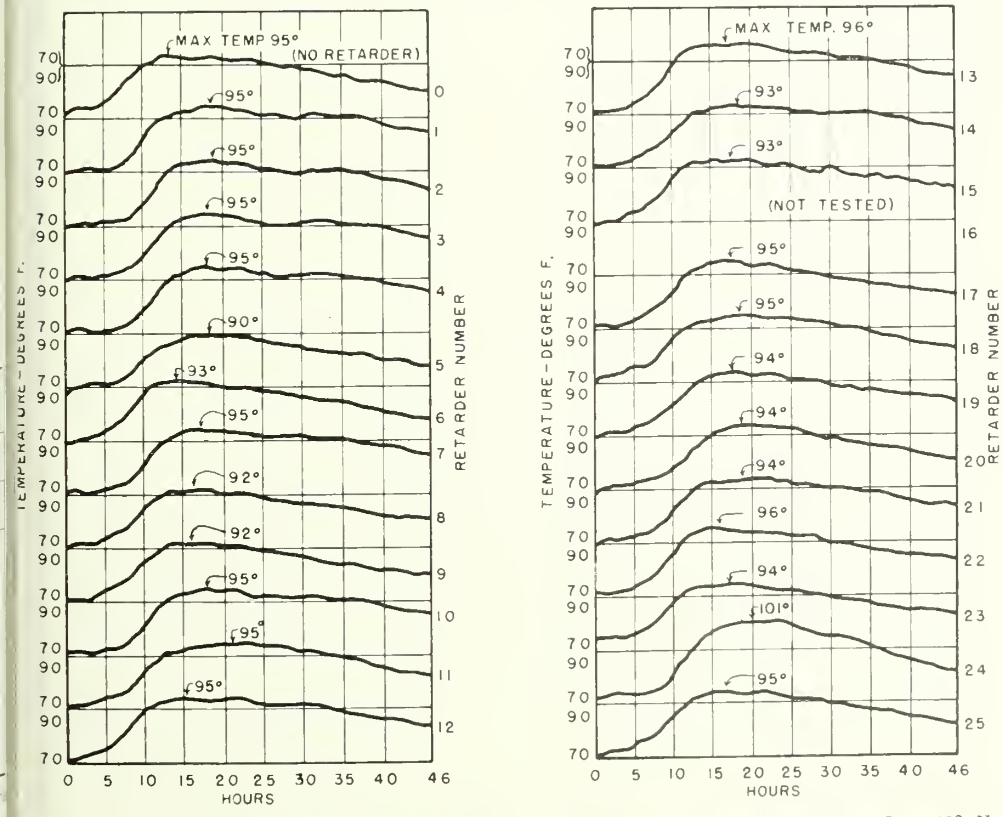


Figure 10.—Effect of overdosage (4 times) of retarder on compressive strength.



igure 11.—Effects of retarder on temperature rise of concrete made and stored at 73° F.

When a retarder was used in sufficient amounts to delay the setting time for 2½ to 3 hours as indicated by a 500 p.s.i. Proctor penetration load, the results obtained are as follows:

- For concrete of equal cement content and equal slump, all retarders tested permitted reductions of the water content of a 6-bag concrete mix from 1 to 12 percent (0.1 to 0.7 gallon per bag of cement) below the water content of similar concrete that was not retarded.
- The chemical composition of retarders generally affected the reduction of water. The sucrose carbohydrate retarder permitted the least reduction of water, 1 percent, while the organic acid retarders permitted 5–7 percent reduction, and the lignosulfonates permitted a reduction of 5–12 percent in water content.
- The time of final hardening of retarded concretes at 4,000 p.s.i. Proctor penetration pressure was less than 11 hours; whereas the time of final hardening of retarded concretes at a pin pullout load of 8 p.s.i. was less than 10 hours.
- Concretes prepared with mix No. 2, except for retarder No. 22, had higher compressive strengths at 3, 7, 28, and 365 days than nonretarded concrete. Increases in strength at 365 days were from 9 to 25 percent. Concrete prepared with retarder No. 22, which had an air content in excess of 8 percent, had compressive strengths which were less at all ages than the strengths of the nonretarded concrete.
- Concretes which were prepared with each of 23 retarders, and which had cement contents of 5.75, 5.5, or 5.25 bags, had equal or higher compressive strengths at all ages than the reference mix with a cement content of 6 bags. Concretes containing 5.25 bags of cement which were prepared with each of two retarders had air contents of more than 8 percent and had lower compressive strengths than the 6-bag nonretarded concrete. Increases in compressive strengths of the other 23 retarded concretes were from 2 to 15 percent at an age of 365 days.
- Flexural strengths of 6-bag retarded concretes, of less than 8 percent air content, were from 2 to 18 percent higher at an age of 7 days than the reference mix flexural strength. At 28 days, the strengths of concrete containing 20 of the retarders were greater than the strengths of nonretarded concrete. The flexural strengths of the specimens which had been moist cured for 15 months were up to 9 percent greater for concretes prepared with each of 14 retarders, and from 1–10 percent lower for 10 retarders than the strengths of the reference mix. The flexural strengths at all ages for the retarded concretes containing over 8 percent entrained air were lower than the nonretarded concrete.
- Flexural strengths of concretes of reduced cement contents (5.25 to 5.75 bags per cubic yard) prepared with each of 17 retarders were, at 7 days, equal to or greater than that of the 6-bag nonretarded concrete. At 28 days, concretes prepared with each of 9 retarders showed increased strengths. When compared

with the strength of 6-bag concrete, 7 samples showed reductions of 2–3 percent at 7 days and 16 samples showed reductions of 1–12 percent in flexural strength at 28 days.

- The sonic moduli of elasticity for concretes of 6-bag retarded concrete (excluding the retarder with the high air content) were higher at 7 and 28 days than the moduli of corresponding nonretarded concrete for all but one retarder.
- The sonic moduli of elasticity for concretes of reduced cement content (5.25 to 5.75 bags) with each of 24 retarders were equal to or higher than the moduli of nonretarded concrete of 6-bag cement content except in 3 samples. Values for static modulus of elasticity, however, were generally lower than those that were determined sonically.
- In 22 cases of 6-bag retarded concrete, delayed vibration increased the compressive strengths at 28 days over that of the nonvibrated concrete. The 28-day compressive strength of retarded concrete with a content of less than 6 bags was equal to or greater than the corresponding nonvibrated concrete. The effect of delayed vibration on 28-day flexural strength was not as beneficial as on compressive strength possibly due to the method of vibration employed. Vibration increased the density of retarded concrete and a reduction of entrained air probably occurred.
- Shrinkage due to a loss of moisture was slightly greater with some retarded concretes than for the nonretarded mix. After 200 days of storage in dry air, reductions in length of 0.044 to 0.049 percent for 6-bag nonretarded concrete and 0.039 to 0.059 percent for retarded concrete were obtained. The increase in length of retarded concretes during 200 days of continuous moist storage was generally only slightly greater than that for concrete prepared without a retarder.
- Relative durability factors, the percentage ratios of the durability factor of the retarded concrete to the durability factor of nonretarded concrete, for 6-bag retarded concretes, when frozen and thawed in water, were 61, 79, and 80 with the use of 3 lignosulfonates and were 91 or greater for the other 22 retarders. Relative durability factors for retarded concrete having a cement content of 5.25 to 5.75 bags varied from 75 to 86 for 5 lignosulfonates and were 90 or greater for the other 20 retarders.
- The average absorptions at 28 days for the nonretarded mix were 5.9, 5.8, and 5.5 percent for the top, middle, and bottom sections, respectively, of a 6- by 12-inch cylinder. For retarded concretes in 6- by 12-inch cylinders, average absorptions at 28 days were 5.7, 5.5, and 5.0 percent for the top, middle, and bottom sections when an organic acid retarder was used, and were 5.5, 5.5, and 5.0 percent, respectively, when lignosulfonates were used. In general, the weight per cubic foot of the retarded concretes was only slightly higher than the weight of the corresponding nonretarded mix.

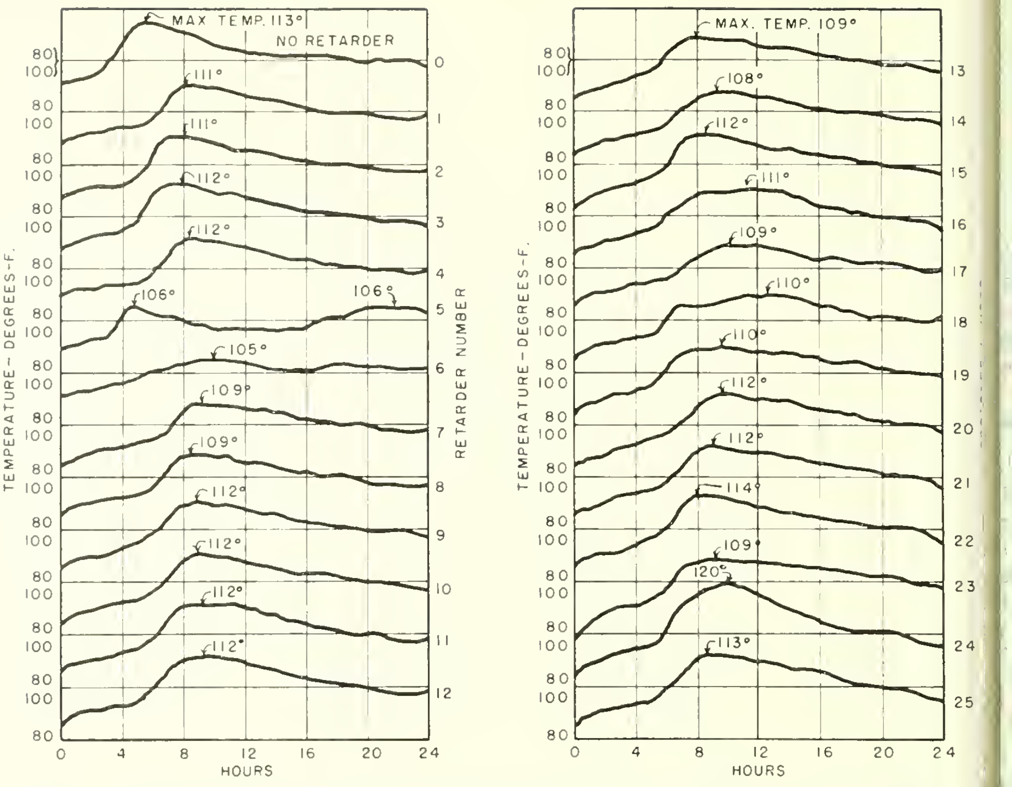


Figure 12.—Effects of retarder on temperature rise of mortar made and stored at 90°

- For 6 of the 19 lignosulfonates, the same amount of retarder caused a retardation of about $2\frac{1}{2}$ hours (at 500 p.s.i. Proctor penetration) whether the concrete was prepared at a temperature of 73° or 90° F. For all other retarders, more material was required to retard the concrete prepared at a temperature of 90° than at a temperature of 73° F. This increase in the amount of retarder needed varied from 33 to 75 percent for the organic acids, from 5 to 37 percent for the lignosulfonates, other than the 6 mentioned above, and was 27 percent for the single carbohydrate retarder. All retarded concrete samples except one developed a final hardening (4,000 p.s.i. Proctor penetration) at a temperature of 90° F. in approximately 7 to 8 hours. Concrete prepared with the other retarder became hard at some time between 9 and 24 hours.
- Concrete prepared with 24 retarders at 90° F. (omitting retarder No. 22) had a 3-day compressive strength that was from 1 to 46 percent greater than the strength of the nonretarded mix. With the use of 2 retarders, 28-day compressive strengths of 96 and 99 percent of the strength of the reference mix were obtained but the other 22 retarders caused strength increases of from 8 to 28 percent.
- An overdosage of 4 times the normal amount of each retarder required to obtain a retardation of $2\frac{1}{2}$ to 3 hours caused setting times of 23 hours to over 150 hours at a Proctor penetration pressure of 4,000 p.s.i. Of 5 lignosulfonate retarders used, 3 caused air contents of over 8 percent and the other 2 required the addition of Vinsol resin for entrainment of 5 percent air. Setting ti of concrete for the overdose of the carb hydrate retarder was 23 hours at a Proct penetration pressure of 4,000 p.s.i. Settin time of concretes (4,000 p.s.i.) with the organic acid retarders varied from 48 hour to over 150 hours.

Compressive strengths at 7 and 28 day obtained for concretes with an overdosage retarders were as follows: Three lignosu fonates (Nos. 3, 7, and 11), which entraine more than 8 percent air, were less than th strength of the reference concrete. At a age of 28 days, the reductions in strengt were from 10–49 percent. Three organ acid retarders (Nos. 4, 5, and 6) were le than the strength of the reference concret after 7 days, but were 3–27 percent high after 28 days. A fourth organic acid retard- (No. 9) caused increases in strength of percent at 7 days and 15 percent at 28 day The carbohydrate sample (No. 1) cause increases in strength over the reference mi of 19 percent at 7 days and 12 percent at days.

- Only retarder No. 24 caused concrete t have a maximum rise in temperature signi cantly higher than that of nonretarded co crete made and stored at 73° F. Developme of the maximum temperature of the oth samples tested occurred approximately 1- hours later than in the concrete without retarder.

In tests of mortar specimens made an stored at a temperature of 90° F., again onl retarder No. 24 caused the maximum rise i

(Continued on page 154)

Recommended Specification for Water-Reducing Retarders for Portland Cement Concrete

. GENERAL

When required by specifications, or special provisions, or permitted by the engineer on vritten request from the contractor, an approved, set-retarding admixture that meets he requirements herein specified shall be added to the concrete.

I. ADMIXTURES

The admixture may be in liquid or powder orm, and of one of the following types: [1]

A. A calcium, sodium, potassium, or ammonium salt of lignosulfonic acid.

B. A hydroxylated carboxylic acid or its alt.

C. A carbohydrate.

II. ACCEPTANCE REQUIREMENTS FOR APPROVAL OF ADMIXTURES

A. *Source of test data for approval of admixure.*—Data from a recognized laboratory that how the water-reducing retarding admixture onforms to the following requirements in this ection shall be submitted. A "recognized" aboratory is any State highway, Bureau of Public Roads, or cement and concrete laboratory, inspected regularly by the Cement Reference Laboratory of the American Society for Testing Materials.

B. *Type of test data.*—The test data shall be btained by the use of concreting materials nd by methods that meet the requirements of urrent standards of the American Association f State Highway Officials (AASHO) or the American Society for Testing Materials (ASTM) as listed in section V of this specification.

C. *Mix requirements.*—The properties of etarded concrete prepared with the admixture nder test, shall be compared with those of a eference concrete, which contains the same ement and aggregates, without retarder, and as the following composition:

Cement content—6.0 ± 0.1 bags per cubic yard;
Air content—5.5 ± 0.5 percent;
Slump—2½ ± ½ inches;
Fine aggregate, by solid volume of total aggregate—36 to 41 percent.

C–1. The mixes of concrete containing the etarder shall have an air content of 5 to 7 percent, inclusive. An approved air-entraining admixture shall be used if the retarder does not entrain sufficient air.

C–2. A sufficient amount of retarding admixture shall be used to cause an increase of 0 to 60 percent in setting time over the setting ime of the reference mix. The setting time for both mixes shall be determined using a Proctor penetration pressure of 500 pounds per square inch at a temperature of 73.4 ± 3° F. for the concretes and ambient air.

D. *Required properties of retarder.*—When added to concrete in powder or liquid form, in the manner prescribed by its manufacturer or marketer and in sufficient amount to retard the setting time 50 to 60 percent, the retarding admixture shall cause the concrete to have the following properties in comparison with those of the reference concrete.

D–1. When the test and reference concrete have equal cement content and equal slump:

(a) The water content shall be decreased at least 5 percent;

(b) The air content of the retarded concrete, with or without an air-entraining admixture, shall not exceed 7 percent;

(c.1) The compressive strength at ages of 3, 7, and 28 days shall be increased at least 10 percent;

(c.2) For concrete to be used in pavement, the flexural strength shall not be reduced at an age of 7 days; at an age of 28 days, the flexural strength shall not be reduced more than 5 percent;

(d) The relative durability factor for the freezing and thawing test shall not be less than 80;

(e) The drying shrinkage at 200 days, after 2 days initial moist curing, shall not increase more than 0.01 percentage points over the shrinkage of the nonretarded concrete.

D–2. When the test concrete has a reduced cement content of ½ bag per cubic yard but the same slump as the reference concrete:

(a) The compressive strength at ages of 3, 7, and 28 days shall not be reduced;

(b) The air content of the retarded concrete, with or without an air-entraining admixture, shall not exceed 7 percent;

(c) The flexural strength shall not be reduced more than 5 percent at an age of 7 days nor more than 10 percent at an age of 28 days;

(d) The relative durability factor for the freezing and thawing test shall not be less than 80.

IV. PERFORMANCE REQUIREMENTS

When a contractor proposes to use an approved water-reducing retarding admixture he shall submit a certificate stating that the admixture is identical in composition with the sample that was used for the acceptance tests. If the admixture varies in concentration from the acceptance sample, a certificate will be required stating that the product is essentially the same for chemical constituents as the approved admixture, and that no other admixture or chemical has been added. Either prior to or at any time during construction, the engineer may require the selected admixture to be tested further to determine its effect on the strength of the concrete. When so tested, 3- and 7-day compressive strengths or 7-day flexural strengths of the concrete, made with the admixture and specified cement and aggregates, in the proportions to be used or being used on the job, shall meet the requirements of Section III D for either equal cement content or reduced cement content. A reference mix of equal or higher cement content, without the retarder, shall be made and tested with the concrete containing the retarder as a basis for comparison.

The strength relations shall be calculated on the results of at least 5 standard 6- by 12-inch cylinders or 5 standard 6- by 6- by 21-inch beams, made, cured, and tested in accordance with the current AASHO Methods T 22, T 97, and T 126, for each class of concrete at each age of test.

V. APPLICABLE SPECIFICATIONS

A. AASHO specification M 85 for portland cement, type I or II.

B. AASHO specification M 6 for fine aggregate for portland cement concrete.

C. AASHO specification M 80 for coarse aggregate for portland cement concrete.

D. AASHO specification M 154 for air-entraining admixtures for concrete.

E. AASHO method T 126 for making and curing concrete compression and flexure test specimens in the laboratory.

F. AASHO method T 119 for slump test for consistency of portland cement concrete.

G. AASHO method T 152 for air content of freshly mixed concrete by the pressure method.

H. ASTM method C 403 for rate of hardening of mortars sieved from concrete mixtures by Proctor penetration resistance needles.

I. AASHO method T 121 for weight per cubic foot, yield, and air content (gravimetric) of concrete.

J. AASHO method T 22 for compressive strength of molded concrete cylinders.

K. AASHO method T 97 for flexural strength of concrete (using simple beam with third-point loading).

L. AASHO method T 160 for volume change of cement mortar or concrete.

M. AASHO method T 161 for resistance of concrete specimens to rapid freezing and thawing in water.

N. ASTM method C 215 for fundamental transverse, longitudinal, and torsional frequencies of concrete specimens.

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[1] Other types of retarders may become available.

Water-Reducing Retarders for Concrete— Physical Tests

(*Continued from page 152*)

temperature to exceed that of mortar prepared without a retarder. Except for retarder No. 5, which developed its maximum temperature of 106° F. at two different times, the other retarders reached their maximum temperatures 3–8 hours later than did the nonretarded mortar mix.

References

(*1*) *Concrete operations in concrete ship program*, by Lewis H. Tuthill. Journal of the American Concrete Institute, vol. 17, No. 3, Jan. 1945, pp. 137–177.

(*2*) *Properties and uses of initially retarded concrete*, by Lewis H. Tuthill and William A. Cordon. Journal of the American Concrete Institute, vol. 27, No. 3, Nov. 1955, pp. 273–286.

(*3*) Discussion of a paper on *Properties and uses of initially retarded concrete*. Journal of the American Concrete Institute, vol. 28, No. 6, Dec. 1956, pp. 1187–1200.

Motor Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in the table on pages 140–141. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of July 1, 1960, have been reviewed for accuracy by the appropriate State Officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit truck, truck-tractor semitrailer combinations, and other combinations.

Editors note: For those who have received distribution of this table prior to this issue, please note the following corrections in footnote references: In the entries for Maryland and Ohio, the footnote numbers 31 and 44 should be deleted from the column "Axle load-pounds, Tandem, Statutory limit"; in the entry for North Dakota, footnote reference 44 should be added in the "5-axle" and "Other combination" columns under the heading "Practical maximum gross weight."

New Publications

Three new publications by the Bureau of Public Roads are now available from the Superintendent of Documents, Government Printing Office, Washington 25, D.C.

Highway Transportation Criteria in Zoning Law, dealing with the zoning law as it relates to highways is available at 35 cents a copy. This publication consists of two articles. The first article, *Highway Transportation Criteria in Zoning Law*, presents a comprehensive annotation of court decisions and statutory law dealing with the use of zoning powers to promote traffic and transportation objectives without hindering progress toward achievement of overall land-use planning goals. The article relates the extent and nature of judicial application of existing highway transportation criteria, and then suggests new criteria for consideration by the courts in handling zoning actions. Finally, the article recommends ways and means to obtain judicial acceptance of new criteria. Included is a summary of pertinent State zoning enabling legislation and a number of suggested zoning ordinance provisions.

The second article presented in this publication, *Police Power and Planning Controls for Arterial Streets*, suggests application of regulatory and zoning measures to traffic problems on urban arterial streets. A Washington, D.C., arterial street is used to illustrate the conflicting purposes and certain manifestations of deterioration sought to be remedied through the utilization of the police powers, access control provisions, and land-use and planning controls.

The second publication available which may be purchased at 20 cents a copy is a revised edition of *The Identification of Rock Types*.

In selecting the best rock for use in different types of highway construction, in planned locations of roads and bridges, and/or in locating sources of aggregates, highway engineers must be able to identify types of rocks in the field without elaborate equipment. This publication, which revises and expands the original edition of 1950, provides a simple method, suitable for field use, for the identification of the different types of rocks. *The Identification of Rock Types* will also be of great use to many other engineers, students, and other persons interested in this subject or whose work requires a limited practical knowledge of geology.

Also available is the publication entitled *Road-User and Property Taxes on Selected Motor Vehicles, 1960*. The 38-page bulletin reports the motor-fuel taxes, passenger car and truck fees, and other road-user taxes and the property taxes (where available) that would be paid on each of 13 selected typical vehicles, ranging from passenger cars to heavy truck-trailer combinations. Assuming certain conditions, such as annual miles traveled, this bulletin reports the amount of taxes paid in each State under the tax schedule in effect on January 1, 1960.

The publication has the same objectives as the three previous reports discussing taxes on selected motor vehicles, the last of which was published in Public Roads in August 1956.

The new study supplies basic data on road-user taxes as of January 1, 1960, and shows the relation of those taxes to property taxes levied on motor vehicles. A second objective of the study was to permit a comparison of the road-user taxes of each State with those of other States on similar vehicles and similar services.

Road-User and Property Taxes on Selected Motor Vehicles, 1960, may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D.C., at 30 cents a copy.

U.S. GOVERNMENT PRINTING OFFICE: 1961

PUBLICATIONS
# of the Bureau of Public Roads

A list of the more important articles in Public Roads *and title sheets for volumes 24–30 are available upon request addressed to Bureau of Public Roads, Washington 25, D.C.*

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D.C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Annual Reports of the Bureau of Public Roads:
1951, 35 cents. 1952, 25 cents. 1955, 25 cents. 1958, 30 cents. 1959, 40 cents. (Other years are now out of print.)

REPORTS TO CONGRESS

A Report of Factors for Use in Apportioning Funds for the National System of Interstate and Defense Highways, House Document No. 300 (1958). Out of print.

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VOL. 31, NO. 7

APRIL 1961

Public Roads

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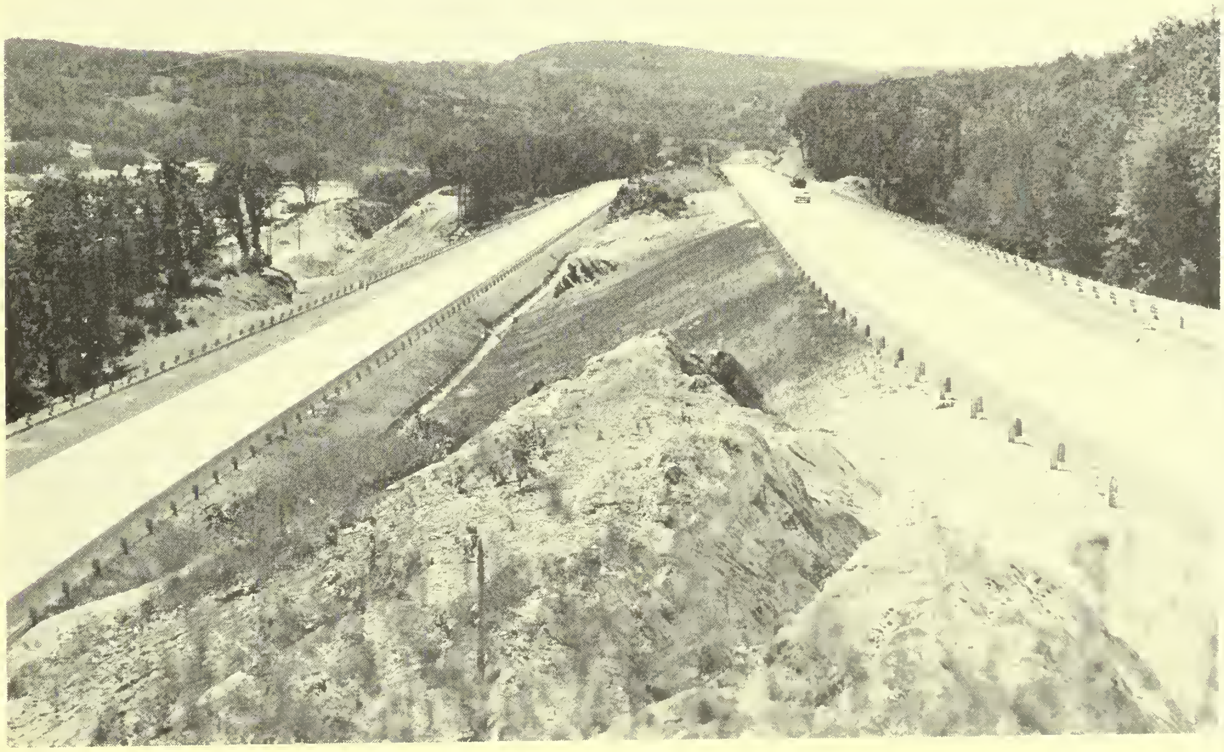

Newly constructed Connecticut Route 8, a Federal-aid primary facility, north of Waterbury.

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LUTHER H. HODGES, Secretary
BUREAU OF PUBLIC ROADS
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Public Roads

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Traffic Approaching Cities

Reported by WALTER G. HANSEN,
Highway Research Engineer

BY THE HIGHWAY PLANNING DIVISION
BUREAU OF PUBLIC ROADS

This article is an extension of work previously done on the subject of the distribution of traffic approaching cities. It confirms previous findings on the distribution of through and central business district traffic and presents the results of an examination concerning the stability of these traffic distributions over a period of years. In addition, it analyzes the distribution of approaching traffic to various concentric rings about the city center and draws several conclusions concerning the general nature of highway facilities required to serve this traffic.

INFORMATION concerning the proportion of traffic bound to and beyond cities was first published by the Bureau of Public Roads in 1944.[1] This information has since been revised and expanded to show the proportion of such traffic destined to the central business district (CBD) as well as the proportion of through traffic, and has appeared in numerous publications during the past decade. The fact that the proportion of through traffic varies inversely with city size is now well recognized and has been a major influence on policy decisions regarding circumferential routes around cities.

In view of the increasing traffic problems in urban areas and the relatively high expenditures required for the construction of urban expressways and arterials, it is especially desirable to reexamine available data to provide up-to-date and, if possible, additional items of information concerning the composition of traffic approaching cities.

Source Material

This analysis is based on traffic information for 193 cities. These data, summarized in table 1, were obtained from studies, conducted by the several State highway departments, which obtained the origin and destination of traffic crossing a cordon line around each of the urban areas. In general, the cordon lines were located at the periphery of the contiguous urban development. Although there was undoubtedly some intraurban or commuter traffic crossing the cordon lines as a result of scattered residential development outside of the study areas, the proportion of such traffic was considered to be small and therefore the preponderance of the traffic studied was intercity or rural-urban in character.

The cities used in this examination varied in size from Davis, Okla., with a population of 1,700, to the Detroit, Mich., metropolitan area with a population of approximately 3 million. The distribution, by size, of the cities used in this study is shown in table 1. City size is based on an estimate of the number of people residing within the cordon area at the time the traffic surveys were conducted. The cities were well distributed throughout the country and the results of this analysis are believed to be representative of average conditions.

The dates at which the individual traffic surveys were conducted varied from 1944 to 1954. A previous, unpublished examination of traffic information for 367 cities indicated that there were no significant trends in the distribution of traffic approaching cities; that is, the proportions of traffic destined to and beyond cities within each population group remained stable over the 10-year period, from 1944 to 1954. A summary of the findings of this examination is shown in table 2.

Based on the findings of the examination, the data used in the remainder of this analysis were aggregated without regard to date of survey. Furthermore, it is felt that the characteristics of traffic approaching cities as presented in this report are fairly representative of existing traffic patterns.

Table 2.—Proportion of total approaching traffic which passes through a city[1]

Population group	Percentage of through traffic by year of survey			
	Prior to 1945	1946–49	1950–54	Average of all surveys
0–5,000	54	48	50	49
5,000–10,000	41	34	50	42
10,000–25,000	34	33	37	35
25,000–50,000	20	25	26	25
50,000–100,000	13	23	18	19
100,000–250,000	15	17	21	17
Over 250,000	10	9	10	9

[1] Based on an analysis of traffic approaching 367 cities.

It should be pointed out that the various presentations in this report represent average or overall conditions and are not necessarily applicable to any specific city. Considerable variation about these average conditions was found in all population groups. The extent of this variation is illustrated in figure 1, which shows the proportion of the total approaching traffic which passes through the individual cities. The curve showing the relationship between city size and the proportion of through traffic is a freehand line based on the weighted average values for the nine population groups shown in table 1. For the most part this is the manner in which the reported relationships were developed.

It is worth noting that separate analyses, not presented here, were performed using the data for the cities in each of five geographical regions. Although there was some variation between the average values and relationships found in the several regions, the deviation of the individual cities about the average values was not significantly improved.

The Analysis

The distribution pattern of highway traffic approaching cities is shown in figure 2. It

Table 1.—Summary of traffic approaching 193 cities

Population group	Number of cities	Average city size	Average daily traffic per city: All roads				Interstate system routes				Other roads			
			Total	Through	CBD	Other local	Total	Through	CBD	Other local	Total	Through	CBD	Other local
0–5,000	11	3,130	7,290	3,930	1,670	1,690	5,320	3,210	1,120	990	1,970	720	550	700
5,000–10,000	29	7,800	8,630	4,520	2,040	2,070	4,730	2,710	990	1,030	3,900	1,810	1,050	1,040
10,000–25,000	43	16,270	11,740	4,520	3,050	4,170	6,310	2,980	1,440	1,890	5,430	1,540	1,610	2,280
25,000–50,000	36	35,780	16,600	4,430	4,110	8,060	7,490	2,590	1,850	3,050	9,110	1,840	2,260	5,010
50,000–100,000	25	78,790	24,600	5,340	5,320	13,940	9,020	2,820	1,910	4,290	15,580	2,520	3,410	9,650
100,000–250,000	31	139,500	32,930	5,890	6,650	20,390	11,490	3,010	2,200	6,280	21,440	2,880	4,450	14,110
250,000–500,000	6	376,700	43,370	4,660	9,260	29,450	17,010	2,250	3,790	10,970	26,360	2,410	5,470	18,480
500,000–1,000,000	9	741,300	63,470	5,470	7,910	50,090	24,650	2,430	2,920	19,300	38,820	3,040	4,990	30,790
Over 1,000,000	3	1,653,000	126,920	13,030	12,110	101,780	27,420	4,040	2,620	20,760	99,500	8,990	9,490	81,020

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[1] Interregional Highways, H. Doc. No. 379, January 1944. See fig. 29 and table 14, p. 60.

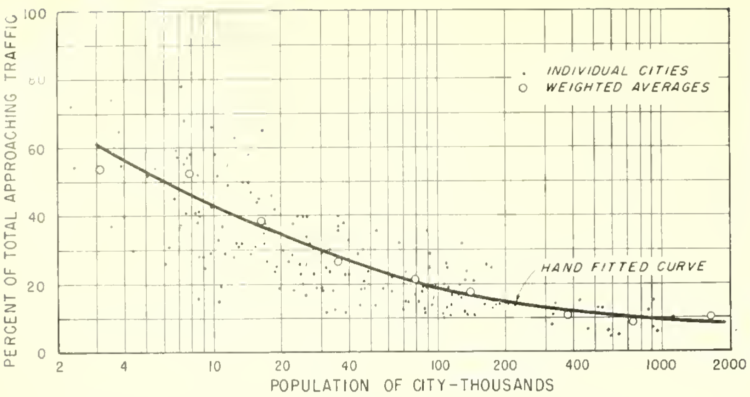

Figure 1.—Percent of traffic passing through a city.

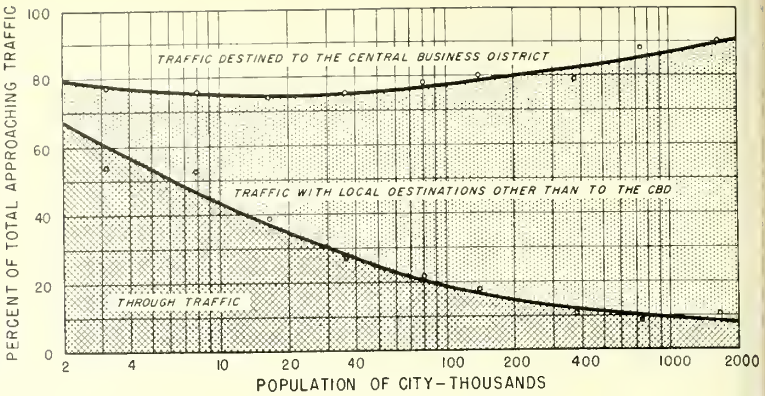

Figure 2.—Distribution of traffic approaching cities.

will be noted that in cities of more than 40,000 population over one-half of the total approaching traffic was destined to local areas other than the CBD. In urban areas of 1 million population, approximately three-quarters of the total traffic was in this category. This traffic is obviously of major significance to highway and transportation planning in medium and larger size cities.

In order to obtain a better understanding of the distribution pattern of this type of traffic, detailed analyses were made of the traffic approaching 11 cities. These cities were selected on the basis of obtaining a range in population size and on the availability of data.

Each of the 11 cities was divided into 4 concentric areas, rings 1–4. These ring areas were centered on the CBD, each with a width of one-fourth of the average distance between the CBD and the cordon line. The proportion of the approaching traffic with destinations in each of these ring areas was then determined for each city. The pattern is illustrated for a typical metropolitan area of 1 million population in figure 3.

The results of this analysis are shown in figure 4. The percentage of the approaching traffic with local destinations to the area bounded by each ring is shown for each of the individual cities. As would be expected, there was considerable variation in results obtained in the 11 cities; however, when arranged by city size, as they are in figure 4, the individual points produced a definite pattern. As city size increased, the proportion of the total traffic destined to the inner concentric area, ring 1, decreased; correspondingly, rings 2, 3, and 4 attracted an increasing proportion of the total trips. This general pattern is approximated by the lines in figure 4 delineating the relative trip attraction of the various rings.

The proportion of traffic destined to the innermost ring decreased from about 60 percent for cities of 100,000 population to 20 percent in cities of 3 million population. The proportion of traffic destined to ring 2 tended to be relatively stable, increasing from 25 percent to about 40 percent over the same population range. The traffic attracted to the outer areas, rings 3 and 4 combined, increased from 15 percent to slightly more than 40 percent. This distribution of traffic with local destination to the various rings was used to modify the information shown in

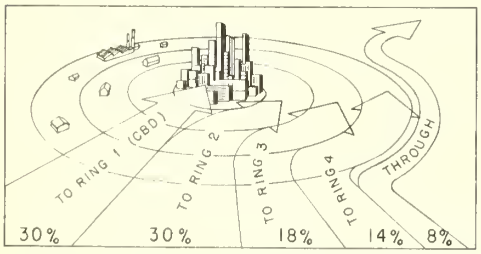

Figure 3.—Distribution of traffic approaching a typical metropolitan area of 1 million population.

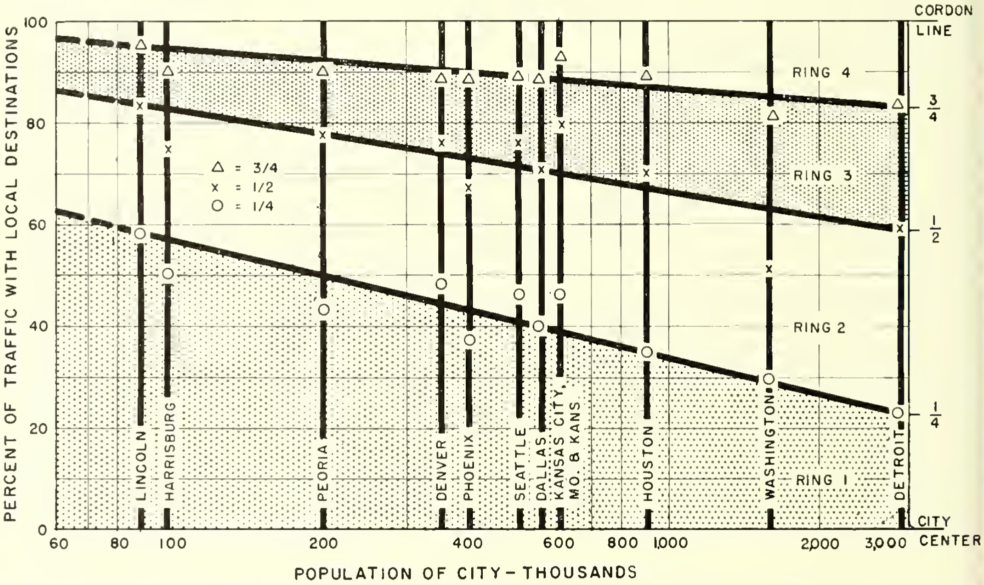

Figure 4.—Local destinations of traffic approaching cities.

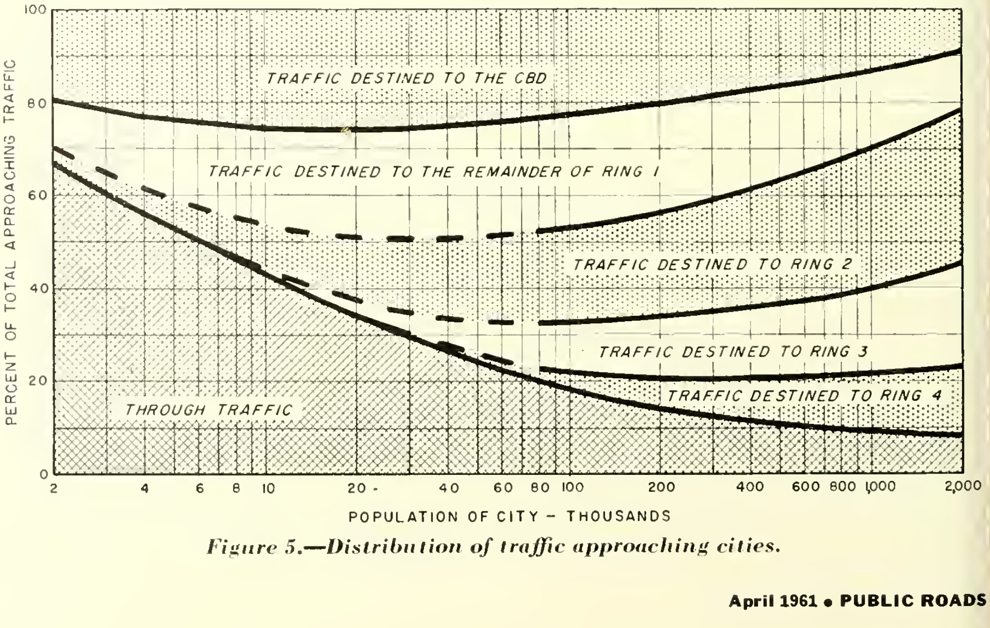

Figure 5.—Distribution of traffic approaching cities.

figure 2. The adjusted chart, indicating the distribution of local as well as through traffic, is shown in figure 5.

The proportion of through traffic, of course, remains as shown in figures 1 and 2. The major point revealed by figure 5 is the dominance of ring 1 (with the exception of the smallest and largest cities), which includes the CBD, as an attractor of traffic from outside the city. In cities between 10,000 and 100,000, this area was the destination of almost one-half of the total approaching traffic. As city size increased to 1 million, this area remained the single most important attractor, although the proportion decreased to about one-third of the approaching traffic.

It is also significant to note the increasing proportion of trips destined to the outer areas, rings 3 and 4, in the larger cities. These areas were the destination of almost one-third of the total traffic approaching cities of 1 million or more.

The proportional distribution of traffic shown in the figures so far presented could be somewhat misleading; therefore, this same distribution is presented in figure 6 in terms of the actual volumes of vehicles involved. This figure shows that the total approaching traffic increased quite rapidly as city size increased; however, the bulk of this increase was made up of vehicles with destinations within the city. The actual volume of through traffic remained relatively stable, increasing only gradually with city size. The volumes of traffic destined through and to each of the local areas in cities of various sizes are shown in the left side of table 3, the values being taken from the curves in figure 6.

Traffic on the Interstate System

A final examination of the data was made to determine whether or not the traffic on roads in locations designated as Interstate System routes exhibited characteristics similar to those found in the previous analysis of the total traffic approaching cities. It should be pointed out that at the time of the origin-destination surveys on which this analysis is based, the Interstate program had not yet been started. The designated Interstate routes have been used in this study because they provided a convenient means of analyzing the traffic using major highway facilities approaching a city, as distinct from all approaching roads.

While the same pattern of a decreasing proportion of through traffic with increasing city size was noted on the Interstate routes, the proportion of through traffic tended to be slightly larger on Interstate routes than on other roads, throughout the range of city sizes. Stated in a slightly different manner, the Interstate routes carried a larger proportion of the total through traffic approaching cities than they did of the total traffic with local destinations. The percentage of each type of traffic carried by the Interstate routes, by city size, is shown in figure 7.

As shown in figure 7, the proportion of the total of each type of traffic that is carried by the Interstate routes decreased as city size increased. For example, in cities of 10,000 population the Interstate routes carried a little over 50 percent of all traffic approaching the city and destined for the CBD; while in cities of 1 million population the corresponding value is about 30 percent. Thus, although the actual volume of traffic on the Interstate routes approaching cities increased with city size, the rate of this increase was much lower than that shown for total approaching traffic in figure 6. The average volume of traffic on the Interstate routes approaching cities of various sizes is shown in the right side of table 3, the values being taken from figures 6 and 7. Of particular interest is the fact that the volume of through traffic carried by the Interstate routes was found to remain almost constant, at approximately 3,000 vehicles per day, throughout all population groups.

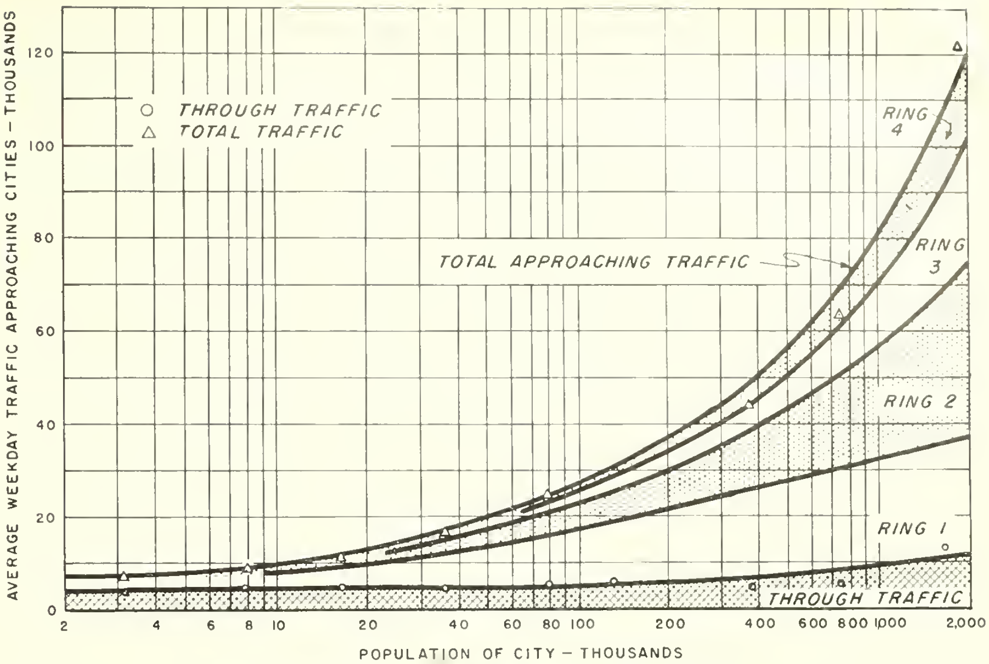

Figure 6.—Destinations of traffic approaching cities.

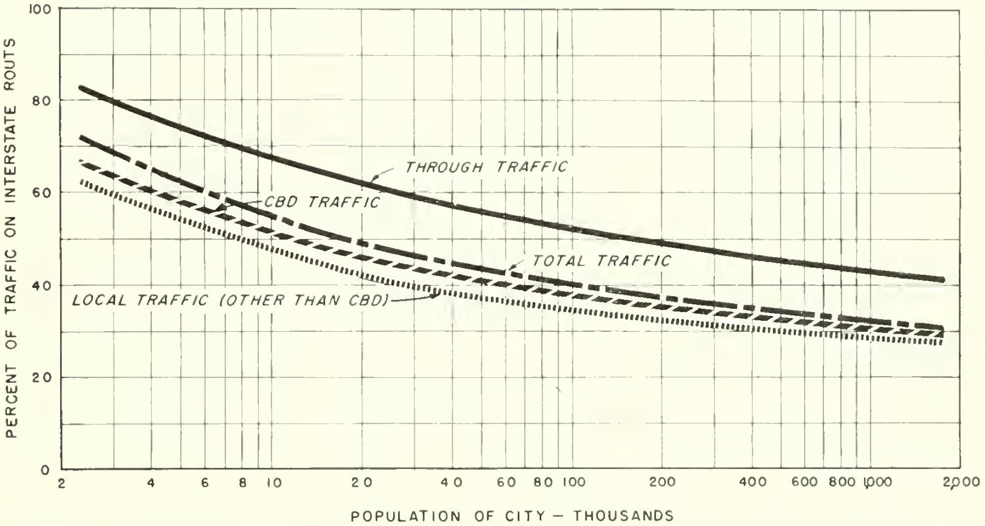

Figure 7.—Proportions of traffic approaching cities on Interstate highways and other roads.

Table 3.—Distribution of traffic on all roads and on Interstate routes approaching cities [1]

City size (population)	Average daily traffic volumes									
	All roads						Interstate routes			
	Total	Through	Ring 1	Ring 2	Ring 3	Ring 4	Total	Through	Ring 1	Rings 2, 3, 4,
10,000	10,000	4,400	4,600	1,000	------	------	5,700	2,900	2,400	400
50,000	20,000	5,000	9,800	3,400	1,800	------	8,800	2,800	4,000	2,000
100,000	27,000	5,100	12,400	5,700	2,700	1,100	10,800	2,700	4,700	3,400
500,000	57,000	6,800	20,500	15,400	9,200	5,100	20,000	3,100	6,800	10,100
1,000,000	82,000	7,400	25,500	24,600	14,700	9,800	26,200	3,200	7,900	15,100

[1] Estimated from figs. 6 and 7.

Conclusions

It would be unwise to attempt to develop recommendations regarding the need for specific highway facilities on the basis of the traffic patterns shown in figures 5, 6, and 7. In the first place, the individual cities exhibited considerable variation from the calculated average conditions; second, and more important, many factors affecting the need for and desirability of a specific type of highway facility serving a community have not entered into this analysis—in particular, local or intraurban traffic needs have not been considered.

However, with the above limitations in mind, the information presented in this report warrants several statements regarding the general nature of highway facilities needed to serve traffic approaching cities.

As city size increases it becomes increasingly desirable to have major highway routes penetrate to the central areas of the cities. The average daily volume of the approaching traffic destined to this area increases from about 5,000 vehicles in cities of 10,000 population to over 20,000 vehicles in cities of 500,000 population. The corresponding volumes of through or bypassable traffic increase from about 4,000 vehicles to almost 7,000 vehicles.

It is not possible to pinpoint the city size at which it becomes desirable to provide penetrating as opposed to bypass routes. It would appear, however, that in the majority of cases this point would be reached in cities of between 25,000 and 50,000 population. In cities ranging upward in size from 8,000 population, the proportion of approaching traffic destined to the central area is greater than the proportion of bypassable traffic; however, the actual volumes of traffic destined to the central area of cities with under 25,000 population are less than 6,000 vehicles per day. In most cases, volumes of this magnitude would not justify the relatively high expenditures required for the construction of major highway facilities to the central areas.

In cities of over 50,000 population, the movement to the central area constitutes the largest single proportion of the total approaching traffic. In terms of actual volume this movement increases from about 10,000 vehicles per day for cities of 50,000 population to more than 25,000 vehicles per day in cities of 1 million or more.

As city size exceeds 100,000 there is an increasing and sizable proportion of the total approaching traffic destined to the peripheral areas of the city. When coupled with the through traffic, this movement would seem to warrant consideration of a circumferential route to act as a distributor for the traffic with local destinations and as a bypass for through traffic. The through traffic plus that local traffic with destinations in peripheral areas (rings 3 and 4) constitutes approximately 40 percent of the total traffic approaching cities of 1 million population, an average daily volume of about 32,000 vehicles. Penetrating routes would, of course, still be required.

Estimated Travel by Motor Vehicles in 1959

BY THE HIGHWAY
PLANNING DIVISION
BUREAU OF PUBLIC ROADS

Reported by ALEXANDER FRENCH,
Highway Research Engineer

MOTOR-VEHICLE TRAVEL in the United States in 1959 totaled 700.5 billion vehicle-miles, an increase of 5 percent over the travel in 1958. Total travel for 1960 is estimated at 720 billion vehicle-miles, a 3-percent increase over 1959, based on information for the first 11 months of the year.

The travel and related information for 1959 is shown in table 1 by road system and vehicle type. Data for Alaska and Hawaii are included for the first time, and a separate tabulation for the 48 contiguous States and the District of Columbia is provided for continuity with data published for preceding years.[1]

The proportions of travel by road system and by vehicle type changed little from 1958 to 1959. Of the 1959 travel, 40 percent was performed on main rural roads comprising 14 percent of the Nation's total of 3.5 million miles of roads and streets. Another 46 percent of the travel was on urban streets, which comprise only 11 percent of the total mileage. Local rural roads accounted for only 14 percent of the travel but make up 75 percent of the total mileage.

Passenger cars represented 83 percent of the vehicles and performed 82 percent of the travel in 1959. Trucks and truck combinations accounted for 16 percent of the vehicles and 17 percent of the travel while similar figures for buses were less than 1 percent.

Average vehicle performance in 1959 differed so little from that reported for 1958 that the changes are not considered significant. The average motor vehicle traveled 9,720 miles in 1959, almost half of it in cities, and consumed 782 gallons of fuel at a rate of 12.43 miles per gallon. The average passenger car traveled 9,529 miles and consumed 666 gallons of fuel, at a rate of 14.30 miles per gallon. A modest increase appeared in average annual mileage traveled and fuel consumed by commercial buses and trucks, but their fuel consumption rates remained about the same as in 1958.

Table 1.—Estimate of motor-vehicle travel in the United States, by vehicle types, in the calendar year 1959

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles per gallon*
50 STATES:[1]										
Passenger cars[2]	219,234	75,919	295,153	277,823	572,976	60,132	9,529	40,056	666	14.30
Buses:										
Commercial	896	148	1,044	1,842	2,886	76	37,974	621	8,171	4.65
School and nonrevenue	602	594	1,196	266	1,462	189	7,735	202	1,069	7.24
All buses	1,498	742	2,240	2,108	4,348	265	16,408	823	3,106	5.28
All passenger vehicles	220,732	76,661	297,393	279,931	577,324	60,397	9,559	40,879	677	14.12
Trucks and combinations	59,661	19,634	79,295	43,859	123,154	11,671	10,552	15,453	1,324	7.97
All motor vehicles	280,393	96,295	376,688	323,790	700,478	72,068	9,720	56,332	782	12.43
48 CONTIGUOUS STATES:[1]										
Passenger cars[2]	218,546	75,817	294,363	276,727	571,090	59,892	9,535	39,923	667	14.30
Buses:										
Commercial	893	147	1,040	1,836	2,876	76	37,842	621	8,171	4.63
School and nonrevenue	600	593	1,193	266	1,459	188	7,761	202	1,074	7.22
All buses	1,493	740	2,233	2,102	4,335	264	16,420	823	3,117	5.27
All passenger vehicles	220,039	76,557	296,596	278,829	575,425	60,156	9,566	40,746	677	14.12
Trucks and combinations	59,479	19,592	79,071	43,675	122,746	11,622	10,562	15,411	1,326	7.96
All motor vehicles	279,518	96,149	375,667	322,504	698,171	71,778	9,727	56,157	782	12.43

[1] Includes the District of Columbia.
[2] Includes taxicabs and motorcycles. There were 565,000 motorcycles registered in the 50 States, and 561,000 registered in the 48 contiguous States.

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[1] See previous articles on motor-vehicle travel data in PUBLIC ROADS; the most recent article, for 1958 data, appears in vol. 30, No. 12, February 1960.

Evaluation of Phosphoric Acid for the Stabilization of Fine-Grained Plastic Soils

BY THE PHYSICAL RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported [1] by JAMES A. KELLEY and EARL B. KINTER,
Highway Research Engineers

This article reports a laboratory study of the effects of phosphoric acid and an amine, in small percentages, on the engineering properties of six fine-grained plastic soils. Pronounced increases in unconfined compressive strength of soaked specimens were obtained for most of the soils when treated with acid alone, and slight additional increases in strength resulted when the amine was also used. In general, plastic properties were only moderately affected by the additives, and there was little change in moisture-density relations. Volume change was reduced to a satisfactory level for five of the soils.

Addition of 2 percent of phosphoric acid resulted in satisfactory compressive strengths for two soils, neither of which had any strength before treatment. Compressive strengths were considerably increased for two other soils by this rate of acid, but not to a satisfactory level. Strength gains for the remaining two soils were not of practical magnitude. The gains for four of the six soils tested were quite sufficient to indicate that phosphoric acid offers considerable promise as a stabilizer for a wide variety of fine-grained plastic soils.

IN 1954 the Bureau of Public Roads, in an effort to stimulate interest in the developnent of chemical products for use in the stabiization of soils, invited chemical manufacturers o aid in the search for suitable chemicals. The ultimate criteria for such chemical dditives are simple: They must be suitable or practical use in road construction, and nust be more effective or cheaper, or both, han the materials already commonly in use, which include portland cement, bitumens, nd lime.

Agreements were made with a number of ompanies under which chemicals would be ubjected to preliminary laboratory testing by he manufacturer and, if found promising, ubmitted to the Bureau of Public Roads for urther laboratory testing and possible subsequent recommendation to State highway departments for full-scale trials in the field. As one result of this program, phosphoric acid was proposed by the Monsanto Chemical Co. and has been tested for use in the stabilization of fine-grained plastic soils.

This article presents some of the results of the laboratory evaluation of the effects of phosphoric acid on six fine-grained plastic soils, conducted by the Bureau of Public Roads between 1955 and the present time.

Conclusions

The Public Roads experiments showed that phosphoric acid produced marked increases in the unconfined compressive strength of two fine-grained plastic soils, the addition of 2 percent of the acid bringing the strength of these soils to over 100 p.s.i., a level considered satisfactory for practical stabilization. With four other fine-grained plastic soils, treated with 2 percent of the acid, compressive strength was not developed to a satisfactory level.

In the one experimental soil containing calcium carbonate, the acid was not effective in strength development.

The addition of phosphoric acid had varying effects on properties other than compressive strength. Values of volume change and moisture absorption were generally brought to satisfactory levels, but changes in moisture-density relations were slight. The liquid limit was greatly lowered for some soils, but for others there was little effect. Plastic limits were practically unchanged. Consequently, changes in the plasticity index corresponded closely to changes in the liquid limit.

For most of the soils used, the addition of an amine compound to the phosphoric acid moderately increased the compressive strength and produced moderate decreases in the liquid limit. In the case of one highly plastic soil, however, the addition of amine caused a pronounced decrease in the liquid limit.

It was found for one soil that the effectiveness of the acid in increasing compressive strength was appreciably reduced by too long an interval between the mixing and the compaction operations.

Other Studies

During the time that the Bureau of Public Roads has pursued its laboratory evaluation of the effectiveness of phosphoric acid in soil strength development, other researchers have also conducted studies of the material and some of their reports have been published. The successful use of phosphoric acid with one fine-grained plastic soil was first reported in 1957 by J. W. Lyons (*1*),[2] who obtained unconfined compressive strengths ranging from 110 to 200 p.s.i. (depending on the length of the curing period) for compacted soil specimens containing about 2 percent of the acid. Lyons also found that the addition of 0.5 percent or less of a compound, identified as amine ODT, facilitated mixing, practically eliminated the critical dependence of the strength of the soil on the water content at the time of compaction, and also rendered the soil water-resistant early in the curing period.

Michaels, Williams, and Randolph (*2*), working with five soils of varying texture, also obtained substantial increases in unconfined compressive strengths as a result of treatment with phosphoric acid or its equivalent of phosphoric anhydride, and with a number of other acidic phosphoric compounds. These three authors also found that a small amount of a primary aliphatic amine would accelerate curing and improve strength retention upon immersion of the specimen in water, although ultimate strengths were somewhat reduced.

Michaels and Tausch (*3*), in studying supplementary additives for improving or reducing the cost of the stabilization of soils with phosphoric acid, found orthorhombic phosphorous pentoxide and phosphate rock plus sulfuric acid to be effective. They also found that, supplemental to phosphoric acid treatment, ferric chloride and octylamine were effective in waterproofing soils.

Demirel, Benn, and Davidson (*4*) found that phosphoric acid increased the unconfined compressive strengths of a number of soils, and showed that calcium carbonate in soils reduced or eliminated strength gains because of the consumption of the phosphoric acid by the carbonate.

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[1] Presented at the 40th Annual Meeting of the Highway esearch Board, Washington, D.C., January 1961.

[2] Italic numbers in parentheses refer to the list of references on page 161.

Public Roads Experiments

The experimental work by the Bureau of Public Roads reported here was conducted with two grades of liquid phosphoric acid supplied by the Monsanto Chemical Co. One of these was a reagent grade furnace acid, containing 85 percent H_3PO_4; the other was a "wet process" acid, containing 40 to 45 percent H_3PO_4. In preliminary tests the wet process acid was found to produce higher values of unconfined compressive strength in soil-acid mixtures than the reagent grade acid, and the wet process acid was therefore used exclusively in subsequent testing. An organic compound identified as SA-4 amine, used as a supplementary additive, was also supplied by the Monsanto Chemical Co.

Six soils—Penn, Jordan, Keyport, Pierre, Hagerstown, and Sassafras—were selected for the experiments. Their characteristics are shown in table 1. As indicated in the table, these fine-grained plastic soils contained a wide range of clay minerals, but are not necessarily considered as representative of all clay soils.

The several tests applied to the raw soils are listed in table 2. These tests, except for the mechanical analysis test, were also applied to the soil-acid mixtures. Table 2 also lists the test values selected as criteria to evaluate whether a given soil had been satisfactorily improved by the stabilizing treatment with phosphoric acid and amine additives.

Mixtures of soil, water, and the chemicals were prepared with a mechanical mixer. Air-dry soil was placed in the mixer bowl and the calculated percentage of chemical (based on dry weight of soil), in water solution, was slowly added while the mixer was in operation. The mixing time was 5 minutes. Atterberg limit tests were performed on a portion of the mixture cured in a high-humidity chamber at room temperature for 8 days.

Volume change specimens were made and tested in accordance with AASHO T 116. Triplicate cylindrical test specimens, 2 inches in diameter and 4 inches in length, were compacted at maximum dry density and optimum liquid content.

Moisture absorption and unconfined compressive strength were measured for each of these specimens after curing for 5 days at room temperature and high humidity, followed by immersion in water for 2 days. If an immersed specimen showed no evidence of slaking, it was surface dried and weighed to determine the moisture absorption. If complete disintegration of the specimen occurred, measurements of both moisture absorption and compressive strength obviously were impossible. If there was more than a trace of slaking, the moisture absorption could not be accurately measured. After soaking and weighing, the usable specimens were tested for unconfined compressive strength.

The 2- by 4-inch cylindrical specimens were prepared by the use of an impact compaction device very similar to that developed by the U.S. Waterways Experiment Station (5). This device employs a 4-pound hammer with a 12-inch drop. Depending on the type of soil being tested, from 8 to 14 blows were required to produce specimens having the same maximum density as obtained in the AASHO T 99 standard method.

Results and Discussion

The effects of the stabilization treatment on the unconfined compressive strength of soaked specimens of three soils—Keyport, Jordan, and Penn—are shown by the data in table 3. Phosphoric acid was added at rates of 1, 2, and 3 percent, with and without th

Table 1.—Characteristics of experimental soils

Property	Test data for experimental soils [1]					
	Penn	Jordan	Keyport	Pierre	Hagerstown	Sassafras
Clay mineral	Illite, chlorite.	Montmorillonite.	Montmorillonite.	Montmorillonite.	Kaolinite, illite.	Kaolinite, vermiculite.
pH	5.0	7.6	4.6	7.0	5.4	5.7
Calcium carbonate..percent	0	10	0	0	0	0
Percentage passing—						
No. 10 sieve	100	100	100	100	100	100
No. 40 sieve	81	99	93	99	99	94
No. 200 sieve	62	91	62	98	98	75
Silt (0.05 to 0.005 mm.) percent	28	44	21	34	25	23
Clay (smaller than 0.005 mm.) percent	45	36	36	61	72	49
Liquid limit	48	34	46	79	78	48
Plasticity index	22	10	24	46	48	23
Maximum dry density,[2] lb./cu. ft	110	100	111	92	91	108
Optimum moisture [2]..percent	18	21	17	26	27	19
Classification (AASHO)	A-7-6(11)	A-4(8)	A-7-6(12)	A-7-5(20)	A-7-5(20)	A-7-6(15)

[1] Sample of Penn soil was from C horizon; samples of other soils were from B horizon.
[2] AASHO T 99-57, Method A.

Table 2.—Tests and criteria for evaluating raw soils and soil-acid mixtures

Test	AASHO designation	ASTM designation	Criterion [1]
Mechanical analysis.	T 88	D 422	
Liquid limit	T 89	D 423	Not over 30.
Plastic limit	T 90	D 424	
Plasticity index	T 91	D 424	Not over 6.
Compaction	T 99-57	D 698-58	
Volume change	T 116		Not over 2½ percent.
Moisture absorption.[2]			Not over 2 percent.
Unconfined compressive strength.[3]			100 p.s.i. or more.

[1] Test value for satisfactory material.
[2] See text, p. 160.
[3] Rate of loading in machine=0.05 inch per minute.

Table 3.—Effect of concentration of phosphoric acid alone, and with SA-4 amine additive, on the unconfined compressive strength of three soils

Treatment		Unconfined compressive strength		
Acid	Amine	Keyport	Jordan	Penn
Pct. [1]	*Pct.* [1]	*P.s.i.*	*P.s.i.*	*P.s.i.*
0	0	0	0	0
1	0	75	13	(2)
2	0	113	15	170
3	0	156	14	202
1	0.5	78	(2)	(2)
2	.5	118	34	184
3	.5	165	(2)	226

[1] Based on dry weight of soil (110° C.).
[2] Not determined; insufficient material for testing.

Table 4.—Effect of phosphoric acid and SA-4 amine on the properties of six soils

Soil	Treatment		Unconfined compressive strength	Volume change	Moisture absorption	Optimum moisture	Maximum density	Liquid limit	Plastic limit	Plasticity index
	Acid	Amine								
	Pct.	*Pct.*	*P.s.i.*	*Pct.*	*Pct.*	*Pct.*	*lb./cu. ft.*			
Penn	0	0	0	9.7	(2)	18	111	48	26	22
	2	0	170	(1)	1.0	17	114	(1)	(1)	(1)
	2	.5	184	(1)	0.1	18	111	(1)	(1)	(1)
Jordan	0	0	0	9.0	(2)	20	104	34	24	10
	2	0	15	1.1	2.4	23	101	40	24	16
	2	.5	34	1.5	2.9	23	99	36	25	11
Keyport	0	0	0	9.2	(2)	17	111	46	22	24
	2	0	113	1.5	1.0	15	114	43	22	21
	2	.5	118	.3	1.2	18	110	40	24	16
Pierre	0	0	0	21.7	(2)	26	92	79	33	46
	2	0	2	.7	(2)	24	91	68	34	34
	2	.5	8	1.2	(2)	25	91	56	36	20
	4	0	28	(1)	7.0	24	92	56	35	21
	4	.5	77	(1)	6.0	25	91	50	36	14
Hagerstown	0	0	5	3.9	4.2	27	91	78	30	48
	2	0	37	(1)	.7	26	88	52	30	22
	2	.5	38	(1)	3.9	26	86	51	33	18
Sassafras	0	0	31	1.0	1.0	19	108	48	25	23
	2	0	74	0	.2	20	107	33	22	11
	2	.5	85	0	1.1	20	103	35	25	10

[1] Insufficient amount of material for testing.
[2] Specimen completely or partly disintegrated when immersed.

addition of 0.5 percent of SA-4 amine. Considerable strength was developed in the Keyport and Penn soils, the effect increasing with increasing rates of acid addition. In contrast, very little strength was developed in the Jordan soil. This may be largely attributed to its calcium carbonate content, which was more than sufficient to prevent the desired soil-acid reaction and, therefore, the strength development (*4*). The strengths developed in the Keyport and Penn soils, with 2 percent of acid, more than met the 100 p.s.i. criterion selected as satisfactory. Small additional strength increases resulted from the addition of the amine.

Since satisfactory strengths were developed when 2 percent of the phosphoric acid was added to two of the three soils initially tested, this percentage rate was used for most of the subsequent experiments. The data in table 4 show the effects of the addition of 2 percent of the acid, with and without the addition of 0.5 percent of the amine supplement, on the six soils included in the tests. In the case of the Pierre soil, as shown in the table, data were also obtained for treatment with 4 percent of acid, with and without 0.5 percent of amine.

The effectiveness of the acid varied considerably, depending on the soil and the particular soil property under consideration. Compressive strength was well above the selected criterion of 100 p.s.i. for the treated Penn soil, slightly above for the Keyport soil, slightly below for the Sassafras soil, considerably below for the Jordan and Hagerstown soils, and extremely low (at the 2-percent acid rate) for the Pierre soil.

Volume change and moisture absorption values in most cases were brought to satisfactory levels, by the addition of acid, but maximum density, optimum moisture, and the plastic limit were not greatly affected. The use of acid brought about marked reductions in the liquid limit for the Pierre, Hagerstown, and Sassafras soils, but had little effect on the liquid limit of the Keyport soil and actually raised that of the Jordan soil. Inasmuch as the addition of the acid had little effect on the plastic limit, the changes in plasticity index corresponded closely to the changes in liquid limit.

In the case of the Pierre soil, the reduction of the liquid limit values brought about by the 2-percent rate of acid application was accompanied by a considerable decrease in volume change, but the acid had little effect on other properties of this soil. The use of acid at the 4-percent rate further decreased the liquid limit and increased compressive strength, particularly with the amine added. These pronounced effects on the properties of the Pierre soil were especially notable since this soil contained 61 percent of a high volume change clay (particles smaller than 0.005 mm.).

Supplementing the acid with 0.5 percent of amine appreciably reduced the liquid limit of the Pierre soil, but little change was noted in the plasticity values of any of the other soils. Additions of the amine also increased the compressive strengths of all of the soils tested, but the effect was pronounced only with the Pierre soil. The addition of the amine had little effect on any of the other soil properties studied.

A supplementary experiment was performed with the Keyport soil to determine the importance of the time interval between the preparation of the soil-acid mixture and the molding of the test specimens. Compressive strengths obtained for 15-minute, 6-hour, and 24-hour intervals between mixing and molding were 104, 80, and 70 p.s.i., respectively. These results indicate that for the development of maximum compressive strength, there should be very little delay between the mixing and compaction operations.

References

(*1*) *Stabilizing a problem soil—cheaply,* by J. W. Lyons. Engineering News-Record, vol. 159, No. 7, Aug. 15, 1957, pp. 101–106.

(*2*) *Acidic phosphorous compounds as soil stabilizers,* by A. S. Michaels, P. M. Williams, and K. B. Randolph. Industrial and Engineering Chemistry, vol. 50, No. 6, June 1958, pp. 889–894.

(*3*) *Phosphoric acid stabilization of fine-grained soils: improvements with secondary additives,* by A. S. Michaels and F. W. Tausch, Jr. Highway Research Board, Bulletin No. 282, 1961 (in press).

(*4*) *Use of phosphoric acid in soil stabilization,* by T. Demirel, C. H. Benn, and D. T. Davidson. Highway Research Board, Bulletin No. 282, 1961 (in press).

(*5*) *Resinous water repellents for soils.* U.S. Waterways Experiment Station, Vicksburg, Miss., Technical Memo. No. 217-1, May 1946.

New Publications

Four new publications by the Bureau of Public Roads are now available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C.

Highway Progress, 1960

Available at 35 cents a copy is the Annual Report of the Bureau of Public Roads, Fiscal Year 1960, entitled *Highway Progress, 1960.* The annual report presents the Federal-aid highway programs, including the Interstate System, and the research and activities of the Bureau.

The 1961 Interstate System Cost Estimate

Four reports, requested by the Congress, were submitted by the Bureau of Public Roads to the Congress, through the Secretary of Commerce, in January 1961.

A new estimate of the total cost of completing the National System of Interstate and Defense Highways was reported to Congress on January 13, 1961. The report, titled *The 1961 Interstate System Cost Estimate,* has been printed as House Document No. 49, and may be purchased from the Superintendent of Documents at 20 cents per copy.

The new cost estimate, made by the State highway departments under the guidance of the Bureau of Public Roads, and resulting from almost a full year's effort, covered all Interstate System work not financed as of January 1, 1960, and showed the total of work remaining to be undertaken as $32.9 billion. Since $1.4 billion of the total is expected to be financed as toll facilities or entirely by States and municipalities, the net cost of work remaining to be done with Federal aid as of January 1, 1960, was $31.5 billion.

Prior to January 1, 1960, the estimate cutoff date, $7.2 billion of Federal-aid and State matching funds had already been spent or obligated for Interstate System improvement. Including these previous expenditures, the total cost of completing the system, from the time of its undertaking in July 1956, becomes $41 billion when provision is made for segments of the system not yet finally located, and for administration, highway planning, and research. The estimated actual construction cost of the system is about $1 billion less than that reported to the Congress in January 1958.

Since the Federal Government is paying 90 percent of the cost of completing the Interstate System, the required Federal share of the $41 billion is $37 billion, of which $25.4 billion has been authorized by Federal legislation. Estimated additional Federal authorizations needed to complete the system thus amount to $11.6 billion.

The new cost estimate was reported to the Congress pursuant to section 104(b)5 of Title 23, U.S. Code, which requires periodic estimates to be made. The estimates serve as a basis for apportionment among the States of the Federal-aid Interstate fund authorizations.

The Highway Cost Allocation Study—2 reports

The final report of the 4½-year highway cost allocation study, conducted by the Bureau of Public Roads pursuant to section 210 of the Highway Revenue Act of 1956, was submitted to the Congress on January 13 and 19, 1961.

Parts I–V of the report have been printed in one volume under the title *Final Report of the Highway Cost Allocation Study,* as House Document No. 54, and is available at 70 cents per copy. Subjects of the five parts are: Introduction and summary of findings; general discussion of the study; allocation of Federal-aid highway cost responsibility between private and commercial users and other classes and interests; allocation of tax support of the

(*Continued on page 166, column 3*)

Highway Construction: An Employment Generator

BY THE CONSTRUCTION AND MAINTENANCE DIVISION
BUREAU OF PUBLIC ROADS

Reported by M. B. CHRISTENSEN, Chief,
Construction and Maintenance Division,
and DAVID A. GORMAN, Highway Engineer

Each billion dollars of Federal-aid highway construction generates on-site employment for 48,000 men with a weekly payroll of over $4.6 million. The relative proportions of the various kinds of labor used on highway construction projects do not vary much on the different Federal-aid systems.

Equipment operators as a group take the greatest share of the pay on highway construction projects, but average hourly earnings on steel bridge projects exceed those on all other types of highway work.

Increased productivity on grading projects, resulting from development and widespread use of equipment, has kept the increase in the price index of grading work moderate during the past two decades. On the other hand, structures, on which the usage of equipment is still relatively limited, have had a steep rise in price index.

EACH BILLION DOLLARS of Federal-aid highway work in active construction status generates the equivalent of a full year's on-site employment for approximately 48,000 men with a weekly payroll of about $4,650,000.

Of this working force, there are approximately 18,130 equipment operators and 2,605 equipment repair and servicemen. Weekly earnings of the equipment operators and servicemen total about $2,190,000, or $105 per man. On-site employment also includes 15,605 unskilled laborers, with total weekly earnings of approximately $1,150,000, or about $74 per man; and 11,660 men are engaged in miscellaneous skilled and semiskilled crafts and in professional, managerial, clerical, and service occupations.

These figures are based on data compiled by the Bureau of Public Roads from a survey made in cooperation with the State highway departments in 1958. The survey covered all employment during the 4-week period from July 13 through August 9, 1958, on 3,425 of the more than 6,600 Federal-aid highway construction contracts in active construction status at the time.

The 3,425 construction contracts studied included all types of highway construction and represented all stages of construction ranging from contracts that were just getting started to some nearing completion. A total of 17,033,561 man-hours of labor was used on these construction projects during the survey period; this labor was paid a total of $41,257,904 in wages, including overtime earnings.

Total contract value of the 3,425 contracts was $2,216,343,000. The dollar value of construction work actually performed on these projects during the 4-week period could not readily be determined, but it was estimated to be about $181 million, or about 8 percent of the total contract value. This estimate was based on figures reported to the Bureau indicating that employment on Federal-aid highway construction during 1958 averaged 94,138 man-hours for each million dollars of contract value.

Assuming that each man employed worked an average of 40 hours a week, the 17,033,561 man-hours of labor used during the 4-week period would mean that 106,460 men would have been at work on the construction contracts, which represents an average of 31 men per contract.

A substantial portion of the employees undoubtedly worked more than 40 hours a week inasmuch as many contractors operated on shifts longer than 8 hours a day, and for 6 days a week, in order to expedite the work and to take advantage of good weather, reduced overhead, and other favorable conditions. While this would suggest that fewer men than indicated were actually at work, it seems certain that another substantial percentage of the employees worked fewer than 40 hours a week because of equipment breakdowns, unfavorable weather, materials shortages, or other conditions making it necessary to suspend operations temporarily on all or certain phases of the construction. Assuming that this "undertime" employment of some workmen would approximately offset the "overtime" employment of others, the figure of 40 hours a week appears to be reasonably correct for an average.

Labor Usage in Construction

Employment on individual contracts undoubtedly varies considerably from the average of 31 men per contract because of varying stages of completion of work. On contracts just getting started or nearing completion, the working force is apt to be relatively small; on contracts operating at full capacity a much larger number of men would be at work. The size of the working force also varies considerably according to size of the contract and character of construction work involved.

Equipment operators as a group took the greatest share of pay, 41 percent, and accounted for 38 percent of the man-hours worked. Unskilled occupations accounted for

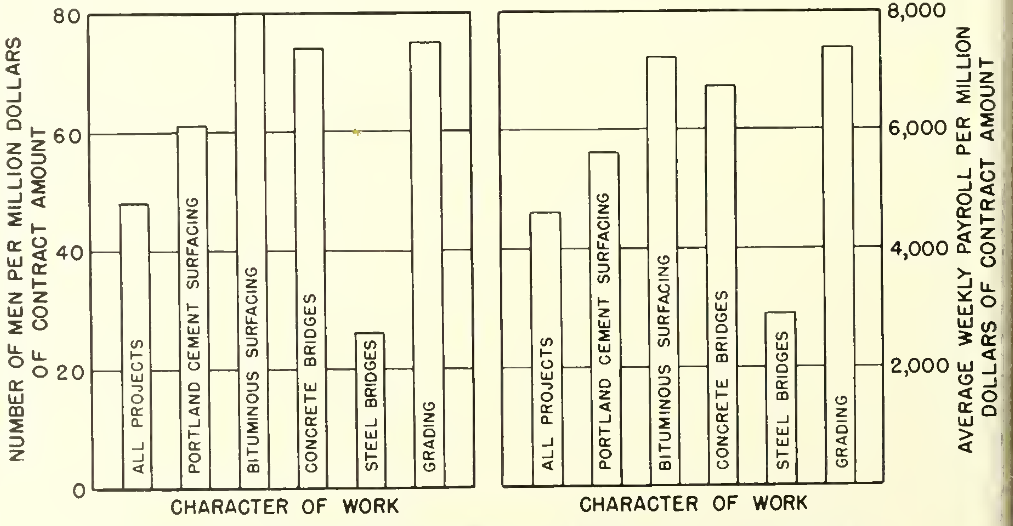

Figure 1.—Labor usage in highway construction on a national basis.

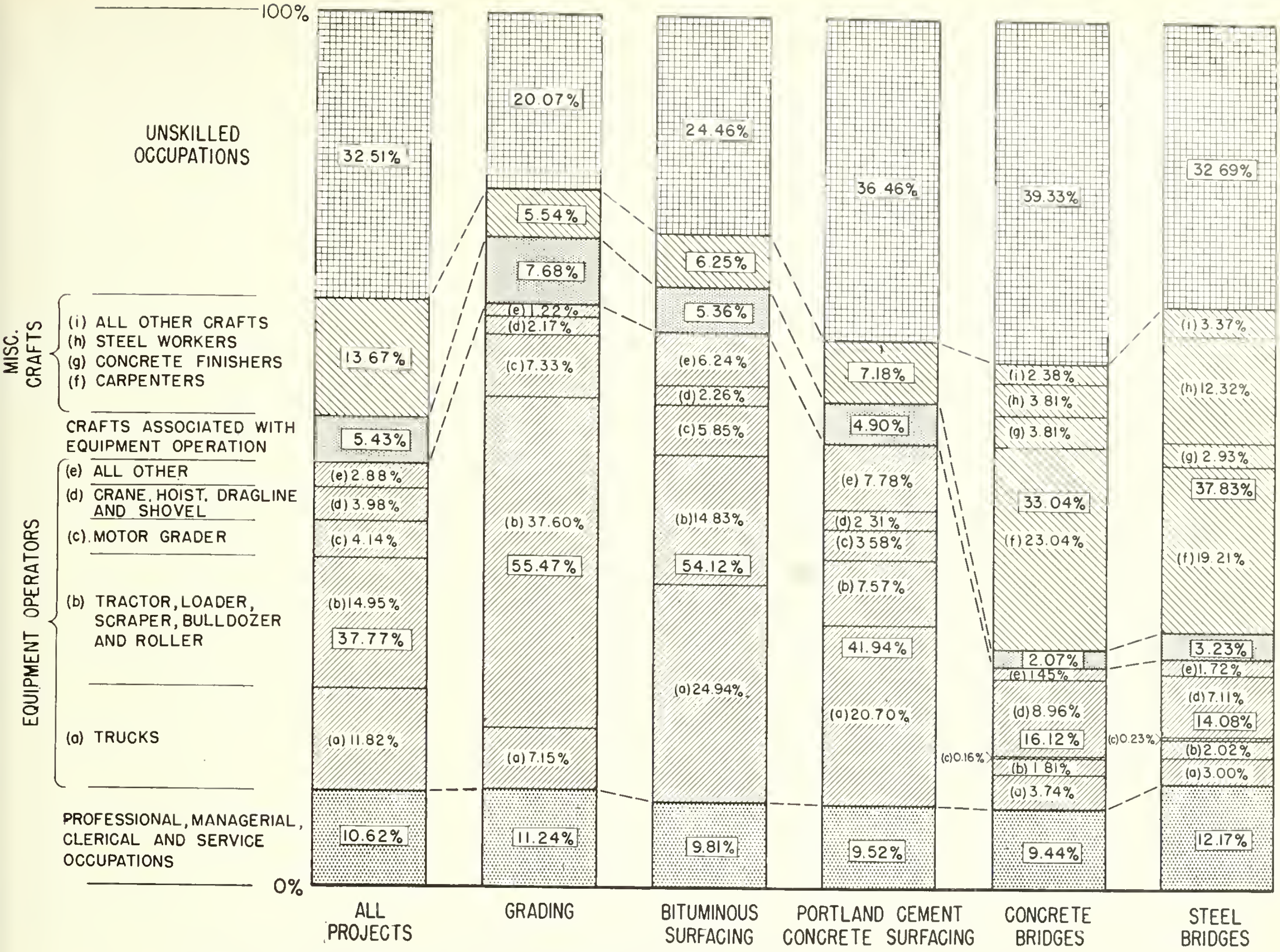

Figure 2.—Distribution of labor employed by contractors on Federal-aid highway construction.

33 percent of the hours worked and 25 percent of the wages paid. Table 3 shows the percentage distribution of hours worked and wages paid by occupational group, and the average hourly earnings, in detail (p. 165).

Job opportunities by type of construction

The number of jobs generated and the employment opportunities of each craft and occupational group was affected by the type of construction. Bituminous surfacing projects generated work for more workers per million dollars of contract amount than any of the other four types of construction shown. On the basis of a 40-hour week, bituminous surfacing projects generated employment for 80 men per million dollars of contract amount, compared to 48 men for all types of construction combined, as indicated in figure 1. Steel bridge projects required the least number of workers per million dollars of contract amount. Figure 1 indicates that for the same contract amount, bituminous surfacing created employment for about one-third more workers (31 percent) than portland cement concrete surfacing projects. It should be recognized that bituminous surfacing represents a broad range of work, from low-type bituminous surface treatment to high-type bituminous concrete.

The type and relative proportions of labor used, as shown in figure 2, differed according to the type of construction. It is noted that except for bridge construction contracts the largest single classification of labor used was equipment operators. On grading work, operators of equipment, together with the associated crafts (mechanics, oilers, firemen, and other workmen who repair and service equipment), represented 63 percent of the total working force. The same groups accounted for 59 percent of those employed in bituminous surfacing, and 47 percent of those in portland cement concrete pavement work. In contrast, workmen engaged in operating, repairing, and servicing equipment accounted for only about 18 percent of the working force on concrete and steel bridges.

The equipment operator classification consisted primarily of four main subclassifications: (1) truckdrivers; (2) operators of tractors with attachments and supplemental units, such as bulldozers, scrapers, loaders, rollers, and rippers; (3) operators of machines such as power shovels, cranes, draglines, hoists, and piledrivers; and (4) motor grader operators. This general classification also includes operators of equipment for: (1) crushing aggregates; (2) batching, mixing, placing, and finishing structural concrete, bituminous surfaces and bases, portland cement concrete bases and pavements, and other types of bases and subbases; and (3) drilling.

As shown in table 3, truckdrivers constituted the largest labor-cost factor among the many classifications of equipment operators employed in highway work. They led not only all other operators but also all other individual occupations, accounting for almost 12 percent of hours worked and 11 percent of wages paid. As shown in figure 2, bridge construction projects generated relatively minor demand for truckdrivers. Truckdrivers on concrete and steel bridge contracts accounted for less than 4 percent of the total time worked. On the other hand, surfacing jobs required a great number of truckdrivers. Truckdrivers accounted for 21 percent of the worktime on portland cement concrete surfacing jobs and 25 percent of the time on bituminous surfacing jobs.

Tractor, grader, scraper, and power crane operators each contributed from about 4 to 5 percent of man-hours worked and collected

Table 1.—Percentage distribution of the total hours worked among the various major occupational groups, on a regional basis

Occupational group	Percentage distribution of hours worked, on a regional basis[1]										
	All regions	Region 1	Region 2	Region 3	Region 4	Region 5	Region 6	Region 7	Region 8	Region 9	Region 10
Professional and managerial occupations	9.40	11.29	11.08	9.05	8.26	9.39	8.01	10.36	10.41	9.08	13.09
Clerical occupations	.92	1.19	1.12	.55	.80	1.01	.53	.99	1.06	1.38	1.71
Service occupations	.30	1.04	.12	.14	.03	.42	.12	.09	.00	.32	.44
Equipment operators	37.77	31.25	34.52	29.53	38.00	45.50	37.44	35.77	53.42	40.83	44.39
Crafts associated with equipment operations	5.43	6.68	3.41	4.02	3.85	5.81	4.85	6.96	4.18	9.17	15.18
Miscellaneous crafts	13.67	15.04	17.88	13.45	15.08	10.99	13.85	19.24	11.22	13.86	8.96
Unskilled occupations	32.51	33.51	31.87	43.26	33.98	26.88	35.20	26.59	19.71	25.36	16.23

[1] For States included in each of the Bureau of Public Roads regions, see inside front cover.

from about 5 to 6 percent of the wages earned (table 3). Roller operators accounted for almost 2 percent of both hours and wages. The remainder of the operator classifications accounted for less than 1 percent each. Equipment operator apprentices were credited with 0.03 percent of man-hours and 0.02 percent of total wages.

Other crafts associated with equipment operation—oilers, mechanics, firemen, stationary engineers, and pile-drivermen—accounted for about 5 percent of hours worked and 6 percent of wages earned. Altogether, operation and care of equipment required 43 percent of the man-hours worked and consumed 47 percent of the payroll.

On a Bureau of Public Roads regional basis, as shown in table 1, contractors in the Northwestern States (region 8) were the largest users of equipment operators in proportion to the total number of hours worked. The percentage distribution of the total time accounted for by equipment operators in region 8 was 53 percent, compared to the national average of 38 percent. The Southeastern States (the Bureau's region 3) ranked the lowest in the proportion of equipment operator time. The percentage, however, was only slightly less than that for the Northeastern States (region 1).

Carpenters were the second largest single occupational group employed on highway construction, accounting for nearly 6 percent of hours worked and 8 percent of wages paid. Within the classification of miscellaneous crafts, carpenters accounted for almost 50 percent of the total hours worked.

Bridge projects produced more job opportunities for carpenters than did any other type of construction. Carpenters accounted for 23 percent of the hours on concrete bridge construction projects and 19 percent of the hours on steel bridge construction projects. On the other hand, surfacing and grading projects offer relatively few job opportunities for carpenters. Carpenters accounted for less than 2 percent of the time on surfacing and grading projects. Structural steel workers and welders naturally found their greatest job opportunities on steel bridge projects, accounting for 10 percent of the total hours.

Unskilled labor represented a substantial percentage of the working force on all types of construction, varying from 20 percent on grading work to 39 percent on concrete bridges. Unskilled labor accounted for a greater share of the total hours worked on highway construction in the Southeastern States (region 3) than anywhere else in the country.

From 9 percent to 12 percent of the working force was made up of professional, managerial, clerical, and service personnel, such as engineers, superintendents, foremen, timekeepers, clerks, cooks, flagmen, and watchmen.

Equipment use and construction costs

The development of new and better construction equipment and its widespread usage on grading and bituminous surfacing projects is indicated in figure 2 by the high proportion of equipment operators. These developments have no doubt had a significant effect upon productivity. The increased productivity, with its resultant savings in construction cost, has been particularly noticeable in the price index for grading. The increase in this price index has been moderate in comparison to the increase in the price index for concrete and steel structures during the past two decades, as shown in figure 3. On the other hand, the relatively small proportion of hours accounted for by equipment operators on bridge projects and the high proportion of hours accounted for by workers in the crafts and unskilled labor categories, approximately 70 percent of the total time, are indicative of the lesser degree of mechanization in structural work with consequent lower productivity for the labor-equipment combination and greater increase in unit costs than for grading and surfacing work. This, plus the increased cost of materials and wages, have apparently been responsible for the steep rise in the price index for structures over the past two decades.

Labor usage on highway systems

The relative proportions of different kinds of labor did not vary greatly for Interstate, primary, and secondary Federal-aid highway

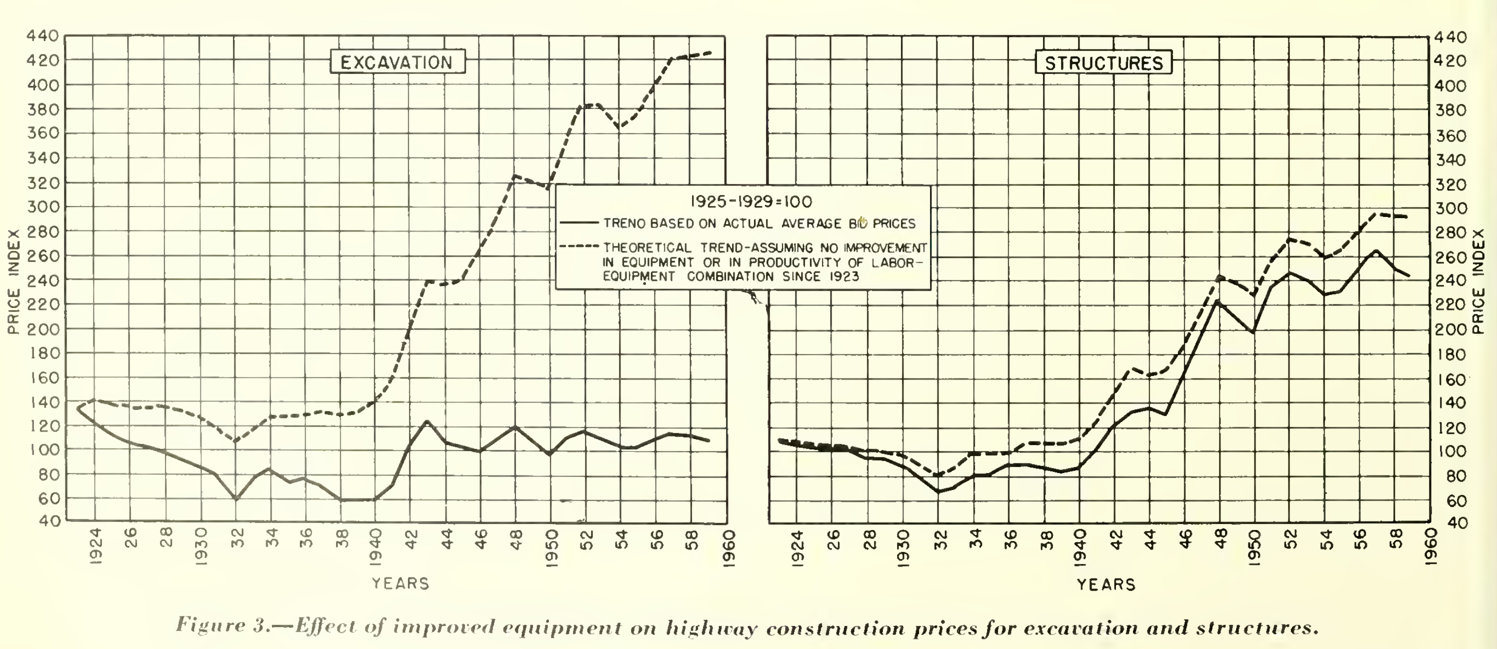

Figure 3.—Effect of improved equipment on highway construction prices for excavation and structures.

Table 2.—Percentage distribution of the total hours worked among the various major occupational groups, on the different Federal-aid highway systems, on a national basis

Occupational group	Percentage distribution of hours worked on—		
	Interstate	Primary	Secondary
Professional and managerial occupations	9.35	9.45	9.39
Clerical occupations	.93	1.02	.71
Service occupations	.24	.38	.27
Equipment operators	34.75	38.10	44.09
Crafts associated with equipment operations	6.06	4.96	5.01
Miscellaneous crafts	16.66	12.82	8.38
Unskilled occupations	32.01	33.27	32.15

projects, as may be seen in table 2. Unskilled labor represented an average of from 32 percent to 33 percent and the administrative and service personnel represented about 10 percent of the working force on each class of highway. The hours worked by equipment operators and associated craftsmen ranged from 41 percent on Interstate projects to 43 percent on primary and 49 percent on secondary projects. The miscellaneous skilled and semiskilled crafts ranged from 17 percent of the hours worked on Interstate projects to 13 percent on primary and to 8 percent on secondary projects.

Average Hourly Earnings

Average hourly earnings based on actual payments made (including overtime) ranged from $1.58 an hour for service personnel such as watchmen and flagmen, to $4.07 an hour for master mechanics, as shown in table 3. Average earnings for unskilled labor were $1.84 an hour. Highly skilled craftsmen such as structural steel workers, electricians, stonemasons, carpenters, plumbers, and operators of heavy, complex equipment were, of course, among those having relatively high hourly earnings. It is possible, however, that a greater percentage of the highly skilled craftsmen's earnings were the result of overtime payments.

On a regional basis (excluding Alaska in which average hourly earnings were unusually high), as shown in figure 4, the average hourly earnings for all workmen combined varied rom a minimum of $1.62 for the Southeastern States (region 3), to a maximum of $3.39 for region 7, the States of Arizona, California, Nevada, and Hawaii. Among the several types of projects (right side of fig. 4), the average hourly earnings on steel bridge projects exceeded those on all other types of work.

It is apparent that an increase in the wages of some crafts could have a very significant effect upon the cost of construction of certain ypes of work if those crafts account for a very arge proportion of the hours worked. On he other hand, wage increases in some crafts may have very little effect upon the overall cost of the work if those crafts account for an nsignificant share of the hours worked. Thus,

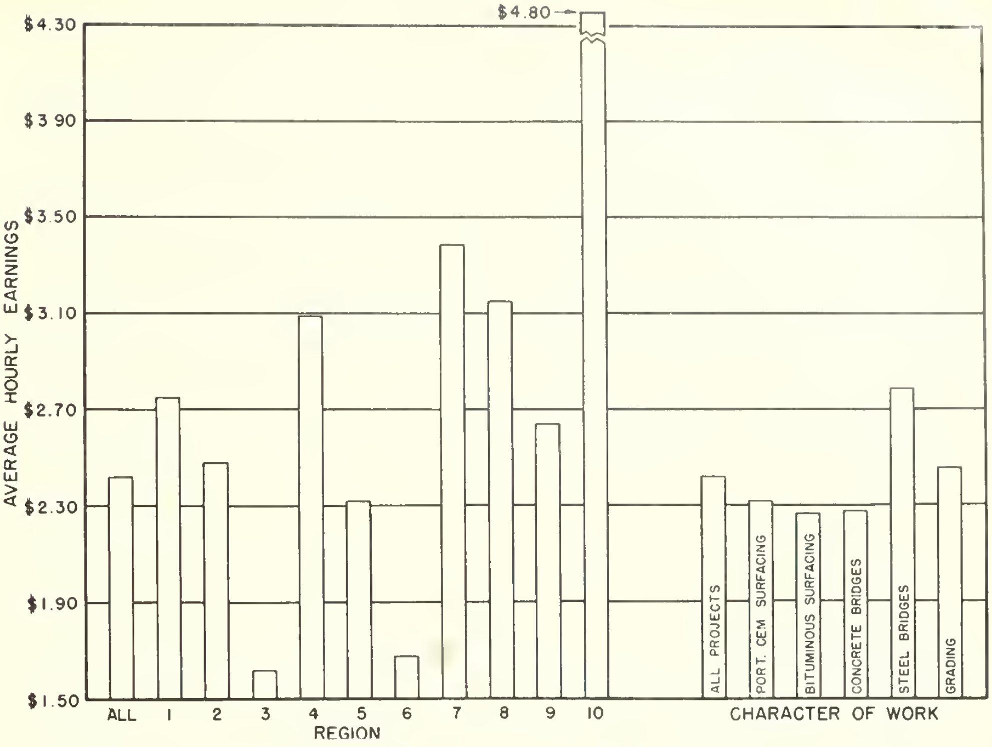

Figure 4.—Average hourly earnings in each region and for each character of work.

Table 3.—Labor usage in highway construction, on a national basis, construction contractor employees only [1]

Occupational group	Percentage distribution		Average hourly earnings [2]
	Hours worked	Wages	
Professional and managerial occupations:			
Foremen	5.98	7.10	$2.89
Superintendents	2.51	3.29	3.18
Civil engineers	.53	.68	3.12
Managerial and official occupations other than superintendents	.20	.27	----------
Office managers	----------	----------	2.48
Project managers	----------	----------	3.42
Surveyors	.17	.19	----------
Instrumentmen	----------	----------	2.31
Accountants	.01	.01	2.33
Subtotal	9.40	11.54	----------
Clerical occupations:			
Paymasters, payroll clerks, and timekeepers	.60	.50	2.04
Clerks, general office	.23	.22	1.97
Bookkeepers and cashiers	.05	.05	2.18
Miscellaneous	.04	.03	----------
Subtotal	.92	.80	----------
Service occupations:			
Watchmen, flagmen, and traffic officers	.29	.19	1.58
Cooks	.01	.01	2.92
Subtotal	.30	.20	----------
Equipment operators:			
Truckdrivers	11.82	10.50	2.23
Tractor operators and loaders	5.20	5.85	----------
Tractor operators	----------	----------	2.53
Front end loader operators	----------	----------	2.59
Motor grader operators	4.14	5.04	2.70
Scraper operators	4.07	4.80	2.62
Crane, hoist, dragline, and shovel operators	3.98	5.09	----------
Crane operators (crawler type)	----------	----------	2.78
Truck crane operators	----------	----------	3.02
Hoist operators	----------	----------	3.56
Dragline operators	----------	----------	2.57
Shovel operators	----------	----------	3.09
Bulldozer operators	3.85	4.57	2.63
Roller operators	1.83	1.78	2.16
Operators of concrete-mixing and concrete paving machines	.67	.67	----------
Concrete mixer operators	----------	----------	2.34
Concrete paver operators	----------	----------	3.21
Operators of asphalt plants and asphalt paving machines	.49	.54	----------
Asphalt plant operators	----------	----------	2.65
Asphalt paver operators	----------	----------	2.38
Batch plant operators	----------	----------	1.74
Concrete and asphalt finishing machine operators	.41	.61	2.47
Rock crusher and gravel plant operators	.40	.50	----------
Crusher operators	----------	----------	2.74
Piledriver operators	.34	.49	3.21
Off-highway hauling equipment operators	.27	.27	2.38

See footnotes at end of table.

(Continued on page 166)

Table 3.—Labor usage in highway construction, on a national basis, construction contractor employees only[1]—Continued

Occupational group	Percentage distribution		Average hourly earnings[2]
	Hours worked	Wages	
Equipment operators—Continued			
Subgrading machine, form grader, and stone spreader operators	0.17	0.17	
Subgrading machine operators			$2.49
Stone spreader operators			2.30
Other construction machinery operators not elsewhere classified[3]	.10	.12	
Trenching machine operators			2.56
Apprentices	.03	.02	
Subtotal	37.77	41.02	
Crafts associated with equipment operation:			
Oilers of machinery	2.38	2.39	2.43
Mechanics	2.50	3.07	
Mechanics			3.05
Master mechanics			4.07
Mechanic helpers			2.18
Firemen	.23	.25	2.43
Engineers, stationary	.22	.30	
Pump operators			3.03
Air compressor operators			3.03
Piledrivermen	.10	.12	3.39
Subtotal	5.43	6.13	
Miscellaneous crafts:			
Carpenters	6.55	7.78	2.86
Concrete finishers	2.06	2.28	
Concrete finishers			2.70
Concrete rubbers			2.18
Drillers and mudjack operators	1.05	1.06	
Drill operators			2.61
Air tool men			1.85
Structural steel workers and welders	1.01	1.42	
Structural steel workers			3.65
Welders			2.76
Reinforcing steel workers	.64	.75	2.86
Metal road form setters and form tamper operators	.56	.47	
Form setters, metal road forms			2.18
Bituminous paving occupations	.44	.42	
Asphalt rakers			2.34
Asphalt distributor operators			2.23
Pipelayers	.32	.27	2.22
Blasters and powdermen	.31	.33	
Blasters			2.53
Painters	.12	.15	3.32
Chainmen and rodmen (surveying)	.09	.07	2.11
Electricians	.08	.12	3.84
Brick and stone masons, and tile setters	.07	.09	
Bricklayers			2.98
Stonemasons			3.32
Apprentices	.05	.05	
Carpenter apprentices			2.30
Ironworker apprentices			2.67
Plumbers	.04	.05	3.26
Other	.28	.23	
Subtotal	13.67	15.54	
Unskilled occupations	32.51	24.77	1.84
Total or average	100.00	100.00	2.42

[1] Based on reports received from 3,425 active Federal-aid projects, covering the 4-week period from July 13 to Aug. 9, 1958.
[2] Includes overtime.
[3] Includes rig operators, dredge levermen, and trenching machine and stabilizer operators.

a significant increase in the hourly wages of truckdrivers could raise the cost of bituminous construction considerably since they account for 25 percent of the hours worked, whereas a wage increase for truckdrivers might have comparatively little effect upon the overall cost of steel bridge construction, where truckdrivers account for only 3 percent of the hours worked. The same principle would apply to carpenters on concrete bridge construction as compared to bituminous surfacing projects.

Although bituminous projects, as mentioned previously, generated employment for more men per million dollars of contract amount than any of the other four types of work, grading projects generated a slightly larger weekly payroll, as shown in figure 1, since average hourly earnings on grading projects exceeded those on bituminous work.

Off-site employment

In addition to on-site employment, discussed in this article, highway construction generates considerable off-site employment. Exact figures were not available, but it is estimated that the off-site employment is at least as great as the on-site employment and may be slightly greater. Off-site employment consists primarily of workmen engaged in the production, processing, and distribution of construction materials and equipment, and also includes employees in construction contractors' central offices, supply depots, and repair shops.

New Publications

(Continued from page 161)

Federal-aid systems among vehicles of different dimensions, weights, and other specifications; and competition of highways with other modes of transportation.

Part VI of the report has been printed under the title *Economic and Social Effects of Highway Improvement*, as House Document No. 72, and is available at 25 cents per copy.

The purpose of the study was to make available to the Congress information on the basis of which it may determine what taxes should be imposed by the United States, and in what amounts, in order to assure an equitable distribution of the tax burden among the various classes of persons using Federal-aid highways or otherwise deriving benefits from them. The report does not contain any statements or recommendations regarding Federal fiscal policy relating to highways.

In considering the problem of cost allocation among various classes of vehicles, four different approaches were undertaken. One of these, relying upon data to be derived from the American Association of State Highway Officials Road Test, cannot be completed until the Road Test data analysis has progressed to a further stage. It is expected that this work will be completed in time to submit definitive findings on this phase of the study to the Congress by midsummer of 1961.

Maximum Desirable Dimensions and Weights of Vehicles

On January 3, 1961, an interim report on the *Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems* was submitted to the Congress pursuant to section 108(k) of the Federal-Aid Highway Act of 1956.

As pointed out in the report, the compilation and analysis of data from the American Association of State Highway Officials Road Test had not been sufficiently advanced to serve as a basis for making specific recommendations, at the time of the report transmittal, as to maximum desirable dimensions and weights of vehicles operated on the Federal-aid highway systems. The report therefore suggested that no changes be made in the existing law on vehicle dimensions and weights (sec. 108(j) of the Federal-Aid Highway Act of 1956), pending submission of a final report containing recommendations. The final report will be made during the summer of 1961.

The report also contained a certain amount of background information on the subject, which presumably will be included in the final report. The interim report has not been printed as a Congressional document, and copies are therefore not available for general distribution.

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Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Final Report of the Highway Cost Allocation Study, House Document No. 54 (1961). 70 cents.

First Progress Report of the Highway Cost Allocation Study, House Document No. 106 (1957). 35 cents.

Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). Out of print.

Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). Out of print.

The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

PUBLICATIONS

Catalog of Highway Bridge Plans (1959). $1.00.

Classification of Motor Vehicles, 1956–57. 75 cents.

Construction of Private Driveways, No. 272 MP (1937). 15 cents.

Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents.

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Financing of Highways by Counties and Local Rural Governments: 1942–51. 75 cents.

General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.

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JUNE 1961

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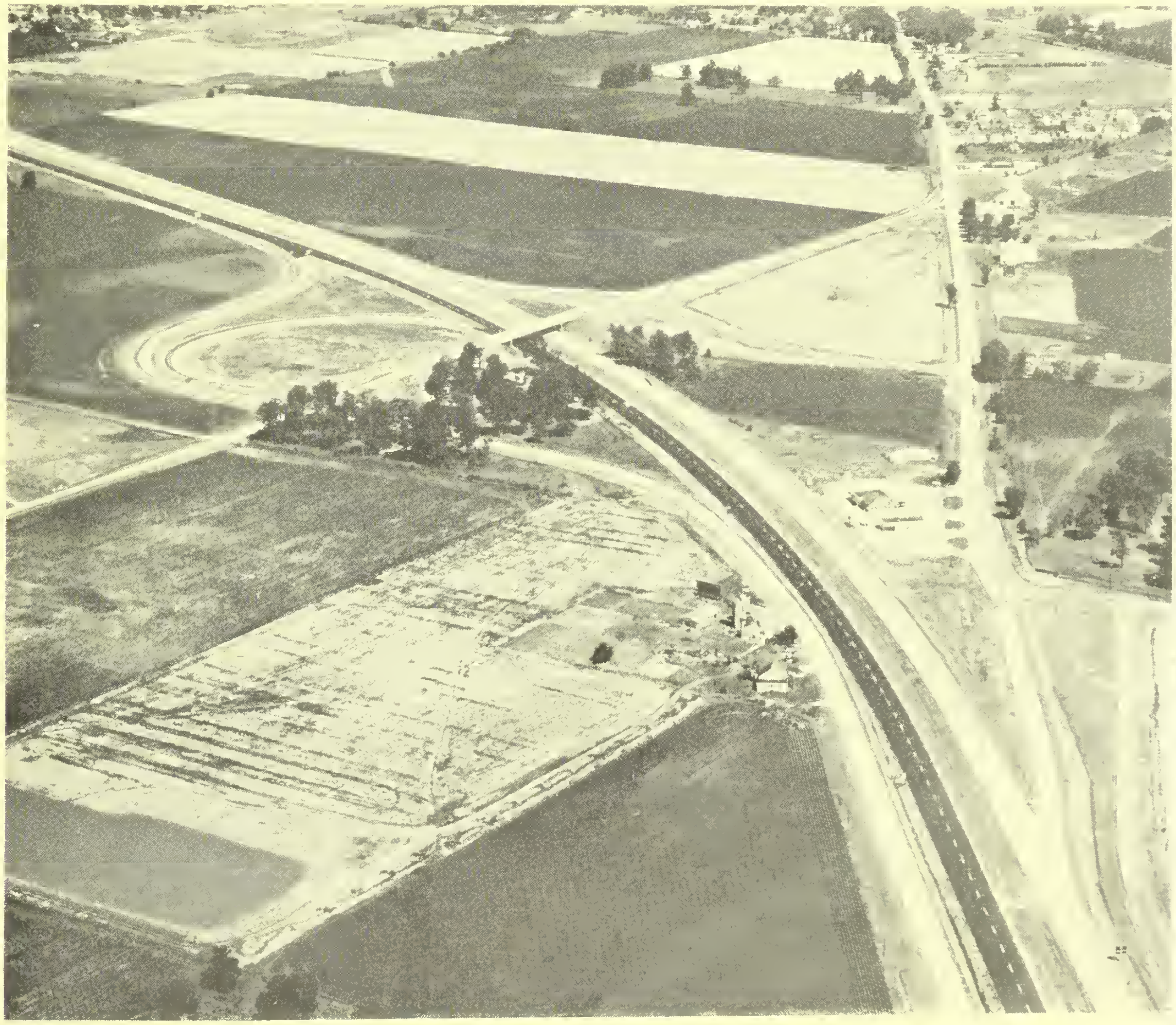

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IN THIS ISSUE:

Characteristics of passenger-car travel for highway user benefit studies.

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E. A. Stromberg, ***Acting Editor***

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Contents of this publication may be reprinted. Mention of source is requested.

Characteristics of Passenger-Car Travel on Toll Roads and Comparable Free Roads for Highway User Benefit Studies

BY THE TRAFFIC OPERATIONS DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by PAUL J. CLAFFEY
Highway Research Engineer[2]

Among the factors important to an accurate analysis of the benefits accruing to passenger-car users through highway improvements are the average overall rates of fuel consumption and speed by type of road, the effects of traffic impedances, the relative attractiveness of the different types of highway improvement benefits to motorists, and the value to motorists of time saving and increased driving comfort. Data useful to passenger-car benefit studies in connection with each of these factors were obtained in 1959 during the operation of a passenger vehicle on 8,000 miles of primary highways in 17 States. These data included rates of fuel consumption, overall speeds, speed changes identified by cause, and records of all traffic impedances.

The study route included 14 sections of toll route where drivers traveling between two particular points have a choice between use of a modern controlled-access toll route or an alternate free route of lesser design quality. At each of these comparison sections, vehicle data were collected for trips on both the toll and free routes. In addition, interview stations were operated on each of the alternate routes to obtain information from the drivers using them.

This report presents the findings of the study, including the average overall rates of fuel consumption and speed on major existing highways and on toll routes and the variation in these overall rates as affected by the frequency of driveways and cross roads, as well as the effects of traffic impedances on passenger-car operation. The report also shows the proportion of passenger-car users electing to use the toll route at each of the 14 toll route comparison sections, and the relative attractiveness to users of the types of benefit realized on both the toll routes and free routes. Finally, the data collected at the toll route comparison sections were subjected to a separate analysis to obtain estimates of the value to motorists of the time saving and increased driving comfort achieved through highway improvements.

Introduction

SOME OF THE FACTORS important to passenger-car benefit studies were investigated by the Bureau of Public Roads during the summer of 1959 by operating a passenger vehicle a distance of 14,000 miles on primary highways in 17 Eastern States and collecting, during this travel, a variety of data relative to passenger-car operation and highway travel characteristics. Included as portions of the study route were 14 locations where a toll road and a major free route are in a position to serve the same traffic. These were taken to represent controlled-access highways of modern design, in the case of the toll roads, and typical primary highways lacking controlled access and some of the other important modern design features, in the case of the corresponding free routes. At each of the 14 locations several comparison runs were made on both the toll road and the free road. At the time of these test runs, interview stations were operated on each route to determine the relative use of the roads and to obtain trip purpose and driver preference data.

Summary of Findings

Fuel consumption and speed

Several types of data useful in analyses of the benefits accruing to passenger-car users through highway improvement were developed in this study. Among these were average overall rates of fuel consumption and speed for operation on existing major 2-lane and 4-lane routes in both urban and rural areas and for operation on toll roads. These data showed that there was little difference between overall speeds on 2-lane roads and 4-lane roads except on main urban routes of large cities outside of the downtown areas where overall speeds on the 4-lane routes were approximately 25 percent greater than on the 2-lane routes. The greatest difference in average overall operating speed observed was between the 60.1 m.p.h. average on rural 4-lane divided controlled-access routes and the 48–50 m.p.h. average on 2- and 4-lane rural routes without control of access.

The fuel consumption rate on the rural 4-lane divided controlled-access routes was about 12 percent greater than on the rural routes without control of access which, in turn, was about 15 percent greater than on urban routes. These differences in fuel consumption rates reflect the over-riding effect of speed on rate of fuel consumption. For the typical traffic volumes carried by existing thoroughfares having no access control, there was little difference either in speeds or in fuel consumption rates for operation on 4-lane routes as compared to operation on 2-lane routes.

Traffic impedances

Certain of the effects of traffic signals, access points (driveways and cross roads without signal protection), and sharp curves on passenger-car operation, together with the average frequencies of these impedances on the 8,000 miles of major routes studied, are included in this report. The study vehicle was required to stop at 43 percent of the traffic lights encountered in urban areas and at 30 percent of those in rural areas. When stopped at traffic lights, there was an average stopped delay of 0.29 minute in urban areas and 0.21 minute in rural areas. Also, the study vehicle was slowed by other vehicles entering or leaving driveways at 1 in every 200 of the driveways encountered in urban areas and at 1 in every 125 of the driveways in rural areas. These data, together with data on speed changes and on the average frequencies of impedances, can be very useful in user benefit computations by providing a means of estimating the effects on traffic operations of highway improvements which eliminate individual impedances or groups of impedances.

---

[1] Presented at the 40th annual meeting of the Highway Research Board, Washington, D.C., January 1961.

[2] Dr. Claffey is also Associate Professor of Civil Engineering, Catholic University, Washington, D.C.

Toll route comparison sections

The relative use of toll routes and alternate free routes by drivers familiar with both routes, and the reasons given by drivers on the compared routes for using either the toll route or the free route, serve to show the relative attractiveness to passenger-car users of the different types of benefits arising from highway improvements. The type of benefit most important to motorists was found to be time saving, with an average of 80 percent of the passenger-car drivers on toll roads and 21 percent of those on free roads stating they selected their travel route on the basis of time saving. The second most important reason given by passenger-car users for selecting their route was greater comfort and convenience. The highway benefits considered of least importance by passenger-car users in their selection of route were found to be greater safety and lower travel costs. Only 13 percent of the free road users indicated that they were influenced not to use the toll road because of the cost factor.

Monetary evaluations

Analysis of the data for comparative toll road and free road trips arrived at an estimate of the average passenger-car user's evaluation of time saving of 2.37 cents per minute of travel time saved, and an estimate of his evaluation of an improvement in driving conditions (measured in units of speed change reduction) of 0.048 cent per 1 m.p.h. reduction in speed change.

The estimate of the value of time saving was of a relatively high order of accuracy; but this was not so for the value of a speed change reduction, largely because there are presumably significant factors affecting driving comfort and convenience which are not reflected in speed changes. The value of each unit of speed change reduction, as computed in this study, is therefore somewhat high for general use as a measure of driving comfort benefit. However, it may be used to estimate driving comfort benefits arising from improvements which reduce the number of speed change units on roads similar to those for which the study data were obtained—primary rural roads without controlled access.

Finally, the study arrived at the following estimates of the comfort and convenience benefits that users receive through given highway improvements: Elimination of a traffic signal stop in rural areas, 4.32 cents; elimination of a sharp curve, 0.72 cent; elimination of a slowdown (for a through vehicle) at an access point, 0.96 cent.

Background for the Study

An accurate determination of the benefits accruing to passenger-car users through highway improvements of various kinds is of paramount importance in highway user benefit studies. The number of passenger cars on the roads and streets and the volume of passenger-car travel accumulated each year presumably make the aggregate benefits from highway improvement for this type of vehicle greater than the combined total for all other types of vehicles.

Among the important factors in passenger-car benefit studies are: (1) Fuel and time consumption both on thoroughfares having numerous traffic signals, access points, and sharp curves, and on divided highways with no traffic signals and fully controlled access, (2) the effect of traffic signals, access points, and curvature on highway vehicle operation, (3) the relative importance to motorists of the various types of benefit accruing through highway improvements, and (4) estimates of the value to the motorists of the time saving and increased driving comfort accruing to users through highway improvements. These items are concerned both with the overall effects of certain types of highway improvement on passenger-car operations and with the values drivers place on improved travel conditions. Numerous other factors having important effects on passenger-car user benefits, such as the relationship between highway design characteristics and accident rates, the effect of surface conditions and vehicle speeds on vehicle maintenance costs, and the value of reduced travel distance, are not included in this study.

Fuel and time consumption in passenger-car operation is affected by several highway factors, including length, relation of capacity to average daily traffic, the frequencies of sharp curves, intersections at grade, and driveway entrances, surface type, and gradients. Some data are currently available on the fuel and time consumption of passenger cars as affected by each of these factors, and several studies have been made of the overall fuel and time consumption of passenger cars operating over limited distances under a particular set of highway conditions (*1*, *2*).[1] However, for benefit studies of large-scale improvement projects more information is needed on overall average speeds and average fuel-consumption rates for operation on typical present-day highways and on highways constructed to the highest design standards.

Predictions of the time and fuel benefits to arise through a general highway improvement program can be made by summing the savings for each item of improvement, such as elimination of intersections at grade and addition of traffic lanes. However, time and fuel savings computed in this manner should be compared with the difference in overall fuel and time consumption of highways of the general type as that involved in a particular analysis, and highways built to high standards, to guard against inadvertently inflating benefits by counting the same items of benefit more than once. Moreover, average overall values of time and fuel consumption for operation on roads which have intersections at grade, frequent access points, and sharp curves, and for operation on divided highways with full control of access, can often be used to make preliminary estimates of the fuel and time savings to result from a particular major highway improvement project.

Traffic impedances such as traffic lights access points, and sharp curves affect vehicl operation by forcing drivers to make undesired stops and slowdowns. These spee changes not only increase fuel and time consumption but are annoying to drivers. As a aid to estimating the extent of the benefit to accrue to motorists through highway improvements that eliminate these impedances information should be available on th frequency of the different types of impedances the average speed changes caused by eac impedance, and, in the case of stops for traffi signals and stop signs, the average duratio of the stopped delays.

An aspect of user benefit analysis of significance in connection with passenger-car benefi studies is the relative preference of users fo the various types of benefit arising throug highway improvement. Information on th relative attractiveness to motorists of reduce travel cost, time saving, greater safety, an increased driving comfort can be of materia assistance in the computation of benefits b providing a guide to the kinds of improvemen most desired by users and to the relativ advantages, from the users' point of view, o the types of benefit realized from thes improvements.

A knowledge of motorists' evaluation o two of the benefits brought about by highwa improvements—time saving and increase driving comfort—is of paramount importanc in passenger-car benefit studies. Many highway improvements, particularly those on large scale in rural areas, bring about highe average operating speeds. Since for the normal range of passenger-car speeds in rura areas the operating costs for fuel, oil, an maintenance increase with increased speed these improvements frequently result in increased operating costs (*3*). Consequently most of the benefits accruing to passenger-ca users through highway improvement are thos associated with time saving, increased drivin comfort, and safety. The benefits to user resulting from reductions in accident rate through road improvements are subject t continuing study and research. The monetary values to users of time saving and increased driving comfort have an importanc in benefit analyses at least as great as accide cost saving, and warrant thorough invest gation.

The Study Operation

The study vehicle

The vehicle used for the study was a 195 six-cylinder, four-door, standard station wago of popular make, equipped with automati shift. It was necessary to use the statio wagon rather than a sedan or other type passenger car in order to provide sufficien interior space to carry the bulky equipmen for recording study data. However, it believed that the data collected are representative of passenger-car operation. Althoug the study vehicle was comparatively new an had been operated only 3,380 miles at th beginning of the study, it was placed on dynamometer and its engine performance wa given a special check immediately preceding the beginning of the trip. All engine defects discovered at this time, however minor, were corrected. In the 2-month study period, during which the vehicle covered over 14,000 miles of travel, all recommendations of the manufacturer in regard to vehicle care and maintenance were strictly adhered to.

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[1] Italic numbers in parentheses represent references, page 176.

The gross weight of the vehicle when loaded with the data collecting equipment and carrying both the vehicle operator and the observer was 4,900 pounds. The frontal cross section of the vehicle itself was 6 feet wide by 5 feet high, the same as for a passenger car of the same make, but the total cross section was increased by an open-top wooden box affixed to the roof of the vehicle to support and protect a gasoline-powered generator, needed to provide electrical power to operate the data-collecting equipment located inside the station wagon. This box added 1½ feet to the vehicle height for almost the full width of its roof. The full cross section of the vehicle when equipped for collecting data was thus 6 feet wide by 6½ feet high.

Data-collecting equipment

The data-collecting equipment (described in detail in the following paragraphs) consisted of an electronic device for measuring distance and speed data, an automatic printer for recording distance, speed, and time data, a code box for manually adding code numbers to the printer record tape, a fuelmeter, and several hand counters.

The items of electronic equipment were interconnected as a unit called the "traffic impedance analyzer" (see fig. 1). The instrument for measuring distance and speed was actuated by a flexible cable connection to the cable of the vehicle's speedometer. The output information from this instrument was directed as a series of electrical impulses into the automatic printer through appropriate electrical connections. The printer recorded automatically at intervals of one second on a strip of paper tape, registering the travel distance to hundredths of a mile (from a fixed point, usually the beginning point of a study run), the vehicle speed to the nearest mile per hour, and the elapsed time in seconds since leaving the initial point of the study run.

A manual code box with 20 push buttons arranged in two columns of 10 buttons each enabled the observer to record any number from 0 through 9 in each of two columns of the printer tape, changing the numbers each second if necessary. A typical sample of the printer tape is shown in figure 2. A recent article in PUBLIC ROADS contains a full description of the traffic impedance analyzer and an explanation of the operation of its component parts (*4*). However, at the time this study was made the automatically recording fuelmeter described in that article was not yet fully developed for use in the traffic impedance analyzer.

Vehicle fuel consumption data were obtained by using a bellows-type fuelmeter connected to the gasoline line of the vehicle between the fuel pump and the carburetor. The instrument was mounted on the right side of the front seat of the vehicle so as to be easily read by the observer who sat on the rear seat. The fuelmeter, which gave fuel consumption readings to the nearest one-sixtieth of a gallon, was read and the data recorded by the observer at each study check point (as described below under test procedure). Fuel consumption data so obtained were frequently checked for accuracy during the study by comparing the difference in fuel readings between successive additions to the fuel tank with the quantity of fuel put into the tank as measured by the gasoline station fuel pump.

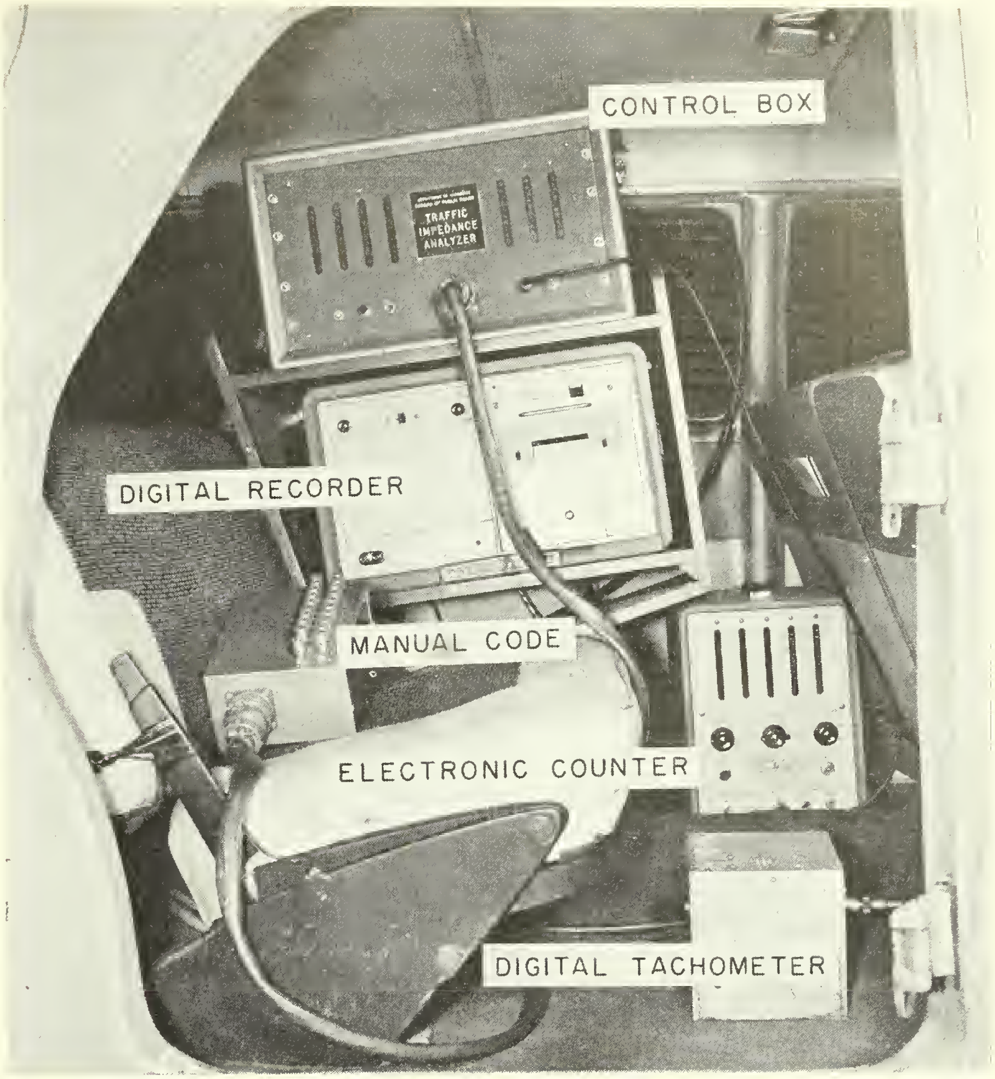


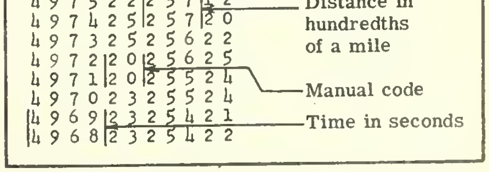
Figure 1.—The traffic impedance analyzer equipment, mounted on the rear seat of the study vehicle.

Study route

The 8,000-mile study route ran north from Washington, D.C., by way of Elizabeth, N.J., to Syracuse, N.Y.; thence east to Boston, Mass., and north to Portland, Maine. From Portland the route ran westward through Syracuse and Buffalo, N.Y., Toledo, Ohio, Elkhart, Ind., Springfield, Ill., and Hannibal, Mo., to Wichita, Kans. From Wichita the route ran south through Tulsa and Oklahoma City, Okla., to Fort Worth, Tex.; thence eastward through New Orleans, La., and along the Gulf coast to Tallahassee, Fla., and southward to West Palm Beach, Fla. From West Palm Beach the route ran northward through Daytona Beach and Jacksonville, Fla., Savannah, Ga., and Richmond, Va., to Washington, D.C. The termini of each of the free road sections of the study route, together with their route numbers, are listed in table 1 in the order in which the data were obtained. All toll road comparison sections are listed in table 2.

Operating procedure

On each section of the route for which data were collected, the study vehicle was operated from one end of the section to the other in a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4 9 7 8 3 2 2 5 9 2 0
4 9 7 7 3 2 2 5 8 1 3
4 9 7 6 2 2 2 5 7 1 8
4 9 7 5 2 2 2 5 7 1 2
4 9 7 4 2 5 2 5 7 2 0
4 9 7 3 2 5 2 5 6 2 2
4 9 7 2 2 0 2 5 6 2 5
4 9 7 1 2 0 2 5 5 2 4
4 9 7 0 2 3 2 5 5 2 4
4 9 6 9 2 3 2 5 4 2 1
4 9 6 8 2 3 2 5 4 2 2
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Figure 2.—Typical record sample from the traffic impedance analyzer.

Table 1.—Free road study sections[1]

No.	Initial point	End point	Route
30	Braman, Okla	Tulsa, Okla	U.S. 177, 77, 75
31	Oklahoma City, Okla	Oklahoma City, Okla.[2]	U.S. 77
32	Moore, Okla	Ardmore, Okla	U.S. 77
33	Fort Worth, Tex	Dallas, Tex	U.S. 80
34	Fort Worth, Tex	Dallas, Tex	Dallas-Fort Worth Tpk.
35	Dallas, Tex	Dallas, Tex.[3]	U.S. 80
36	Shreveport, La	Shreveport, La.[2]	U.S. 80, 71
37	Shreveport, La	Baton Rouge, La	U.S. 71, 190
38	Baton Rouge, La	Baton Rouge, La.[2]	U.S. 61
39	Gonzalez, La	New Orleans, La	U.S. 61
40	New Orleans, La	New Orleans, La.[4]	U.S. 90
41	New Orleans, La	Biloxi, Miss	U.S. 90
42	Biloxi, Miss	Biloxi, Miss.[2]	U.S. 90
43	Biloxi, Miss	Mobile, Ala	U.S. 90
44	Mobile, Ala	Mobile, Ala.[2]	U.S. 90
45	Mobile, Ala	Chipley, Fla	U.S. 90
46	Fort Pierce, Fla	Daytona Beach, Fla	U.S. 1
47	Daytona Beach, Fla	Daytona Beach, Fla.[2]	U.S. 1
48	Jacksonville, Fla	Jacksonville, Fla.[2]	U.S. 1, 17
49	Jacksonville, Fla	Woodbine, Ga	U.S. 17
50	Savannah, Ga	Savannah, Ga.[3]	U.S. 17
51	Hardeeville, S.C	Walterboro, S.C	U.S. 17, 17A
52	Walterboro, S.C	Summerton, S.C	U.S. 15
53	Manning, S.C	Florence, S.C	U.S. 301
54	Rowland, N.C	St. Paul, N.C	U.S. 301
55	Fayetteville, N.C	Fayetteville, N.C.[2]	U.S. 301, 401
56	Raleigh, N.C	Raleigh, N.C.[2]	U.S. 401
57	Raleigh, N.C	Norlina, N.C	U.S. 1
58	South Hill, Va	Petersburg, Va	U.S. 1
59	Petersburg, Va.[5]	Richmond, Va.[5]	U.S. 1
60	Richmond, Va.[5]	Petersburg, Va.[5]	Richmond-Petersburg Tpk.

[1] Sections 34 and 60 are toll roads.

[2] Through city, from city limit to city limit.

[3] From the center of the city to the city limit.

[4] From downtown to the residential area.

[5] For section 59, from south city limit of Petersburg to north city limit of Richmond; for section 60 the same terminals but in the opposite direction.

Table 2.—Free road and toll road comparison study sections[1]

No.	Initial point	End point	Free road route	Toll road route
1	Elizabeth, N.J	Delaware Memorial Bridge	U.S. 1, 130	New Jersey Turnpike.
18	Trenton, N.J	Delaware Memorial Bridge	U.S. 130	New Jersey Turnpike.
19	Camden, N.J	Delaware Memorial Bridge	U.S. 130	New Jersey Turnpike.
2	Syracuse, N.Y	Utica, N.Y	N.Y. 5	New York Thruway.
3	Syracuse, N.Y	Harriman, N.Y	U.S. 81, 11, 17	New York Thruway.
4	Portsmouth, N.H	Massachusetts line	U.S. 1	New Hampshire Turnpike.
51	Portland, Maine	Kittery, Maine	U.S. 1	Maine Turnpike.
6	Toledo, Ohio	Indiana line	U.S. 20	Ohio Turnpike.
7	Elkhart, Ind	Ohio line	U.S. 20	Indiana Toll Road.
8	Wichita, Kans	Wellington, Kans	U.S. 81	Kansas Turnpike.
9	Wichita, Kans	Topeka, Kans	U.S. 81, 50, 75	Kansas Turnpike.
10	Tulsa, Okla	Oklahoma City, Okla	U.S. 66	Turner Turnpike.
20	Tulsa, Okla	Miami, Okla	U.S. 66	Will Rogers Turnpike.
11	West Palm Beach, Fla	Fort Pierce, Fla	U.S. 1	Sunshine State Parkway.

[1] Three trips were made on each pair of comparison routes.

manner as closely typical of the passenger cars in the traffic stream as possible. This was done by having the vehicle "float" with the traffic; that is, operate so as to be passed by about the same number of vehicles as it overtook and passed.

During each test run the traffic impedance analyzer automatically recorded on the printer tape, once each second, the speed, distance, and time. The observer, continuously alert to traffic conditions and highway elements affecting vehicle speed, made use of the manual code box to record on the printer tape, at each speed change, a code number to identify the highway factor or traffic event causing the speed change. The left column of code numbers (on the code box and as recorded on the tape) identified highway and traffic factors such as number of lanes or whether the highway was divided or not; the right column code numbers identified traffic events such as traffic signals or a vehicle suddenly entering from a side road and causing the test vehicle to reduce speed. The complete code used was as follows:

Left column:

1. 2- or 3-lane rural; free-moving traffic.
2. 2-lane rural; trailing another vehicle and unable to pass.
3. 4- or more lane rural without access control.
4. 4- or more lane rural, divided, with access control.
5. 2- or 3-lane urban; free-moving traffic.
6. 4- or more lane urban; free-moving traffic.
7. 2- or 3-lane urban; congested traffic conditions.
8. 4- or more lane urban; congested traffic conditions.
9. 2-, 3-, or 4-lane urban, one-way; free-moving traffic conditions.
0. 2-, 3-, or 4-lane urban, one-way; congested traffic conditions.

Right column:

1. Check point.
2. Traffic signal.
3. Stop sign or flashing red signal, if stopped; otherwise, trailing a truck.
4. Sharp curve or turn, if slowed down; railroad crossing if stopped.
5. Residential driveway where entering or leaving vehicle affected the study vehicle.
6. Commercial driveway where entering or leaving vehicle affected the study vehicle.
7. Overtaking and passing maneuver by study vehicle.
8. Effect of schoolbus in rural areas; or double-parked vehicles in urban areas.
9. Vehicle turning into highway from cross road and affecting the study vehicle.
0. Blank.

The observer manually recorded on a separate data sheet the clock time, vehicle odometer reading, fuelmeter reading, and fuel temperature, at each of several check points on each test run. The check points were places where the character of the highway changed abruptly. For example, each point where a highway entered or left an urban area, even though it was only a small town, was recorded as a check point. Check points were located in this manner to make possible an analysis of the data by type of highway and character of traffic conditions. Check point locations were recorded on the printer tape by using code number 1 in the right-hand code column. Since all check points were selected in advance of the test runs, it was a relatively simple matter to go over the printer tape after completion of each run and write on the tape a complete identification of each check point. The clock time and vehicle odometer reading recorded for each check point constituted a check on the operation of the electronic measuring and recording equipment.

The numbers of access points were recorded for each study section, using hand counters. A separate count was made for cross roads and cross streets, residential driveways, and commercial driveways. Each intersecting highway or street was counted as one cross road regardless of whether it actually crossed the study route or terminated at it. All residential driveways and all entrances and exits to commercial establishments on both sides of the route, on divided as well as undivided highways, were included in the total count of access points. Where a commercial entrance was very wide, each 40 feet of width was counted as one access point. The observer counted commercial and residential driveways with two hand counters, while the vehicle operator counted cross roads with a third hand counter.

Toll route comparison sections

On each of 14 sections of the study route designated as toll route comparison sections, users desiring to travel from one end of the section to the other had a choice between using a major nontoll highway built to standards associated with roads of uncontrolled access and a toll road built to conform with the highest design standards (see table 2).

Table 3.—Average overall speeds and rates of fuel consumption of station wagon on primary routes, by type of route and number of traffic lanes

Type of route	Average overall speed in miles per hour		Average overall rate of fuel consumption			
			In gallons per mile		In miles per gallon	
	2-lane route	4-lane route	2-lane route	4-lane route	2-lane route	4-lane route
Controlled-access rural divided highways	------	60. 1	-- --	0. 09	--	11. 1
Routes without controlled access:						
Rural roads [1]	49. 7	47. 8	0. 08	. 08	12. 5	12. 5
Main urban routes in large cities:						
Downtown areas	23. 0	24. 3	. 08	. 07	12. 5	14. 3
Outside downtown areas	24. 9	31. 1	. 07	. 07	14. 3	14. 3
Main urban routes in small towns	29. 6	27. 2	. 07	. 07	14. 3	14. 3

[1] Exclusive of small towns.

At each of these sections a special study was carried out to obtain directly comparable data of passenger-car operations on the toll roads and the alternate free routes. For each comparison section the termini were common to both routes. In most cases, the toll road routes included short sections of free route at each end to connect the toll routes to the free routes at the common end points.

At each toll route comparison section three test runs were made, traveling on the free road from the designated initial point to the end point and returning on the toll road. Special runs made on the New York Thruway, where the most severe terrain conditions were found, demonstrated that there was no significant difference in the toll road data by direction of travel. The operation of the vehicle on test runs and the kinds of data collected on each run were the same as described above for all test runs, except that on the toll road the electronic recording equipment was operated for only one toll road test run at each comparison section. It was not considered necessary to the conduct of the study to operate the traffic impedance analyzer to record speed changes for more than one toll road run because of the inherent uniformity of speeds encountered in toll road operations.

Analysis of Study Vehicle Data

Four separate analyses were made of the data collected: (1) A comparison of average overall speeds and rates of fuel consumption of a passenger vehicle operating on highways with the highest design standard (toll roads) as compared with operation on major thoroughfares without access control and without many of the other modern design features; (2) determination of the effect on the passenger-vehicle operation on major thoroughfares, of traffic signals, access points, and sharp curves; (3) an analysis of the relative use of toll and free routes; and (4) an investigation of the average motorist's evaluation of the time saving and improved driving comfort resulting from highway improvement.

Fuel and time consumption

The overall average speed and fuel consumption rate of the study vehicle were determined for the distance between each successive check point of each section of the study route. These were computed from the recorded elapsed time, fuel consumption, and distance data. The speeds and rates of fuel consumption computed for all portions of highway having the same general travel characteristics were then grouped together and the average values found for each group—rural divided controlled-access highways (the toll roads); and primary routes in rural areas, in urban downtown areas, in urban areas outside the downtown area, and in small towns.

The results, presented in table 3, demonstrate the general overall effects, on vehicle speeds and rates of fuel consumption, of those improvements which would consist of an increase in the number of traffic lanes from two to four, and of those improvements which bring about the upgrading of the typical primary highway of uncontrolled access to the level of routes designed to the highest standards (toll roads). The speeds and rates of fuel consumption on rural routes were about the same for both 2-lane and 4-lane roads but both were higher on toll routes as compared to free routes for the typical traffic volumes using the routes. The slightly lower average overall speed shown for 4-lane rural roads with no control of access as compared to 2-lane rural roads was undoubtedly due to the much higher traffic volumes encountered on the 4-lane roads.

On free routes in urban areas, except in small towns, the average speeds were greater on 4-lane than on 2-lane roads, but in one case the fuel consumption rates were about the same. In small towns the average overall speed was higher for 2-lane roads than for 4-lane roads, probably because of the greater frequency of traffic signal stops on 4-lane routes (see table 4). The values in table 3 provide general indications but are inconclusive for direct computation of user benefits since they do not differentiate according to traffic volumes. They are useful, however, as overall checks on time and fuel benefits computed by other means.

Effect of driveways and cross roads

The variation in the average overall speeds and fuel consumption rates of the study vehicle as related to the frequency of driveways and nonsignalized intersections (cross roads) on primary 2-lane rural roads is shown in figure 3. This figure may be used to estimate the effects on passenger-car time and fuel consumption to result through improvements reducing the frequency of access points on primary 2-lane rural roads for the ranges of average daily traffic volumes typical of such roads.

Figure 3 shows that where there were fewer than two cross roads per mile, both the average overall speed and the average rate of fuel consumption decreased with an increase in the frequency of driveways from less than 10 to 10–20 driveways per mile. However, only the average overall speed continued to decrease when the frequency of driveways was more than 20 per mile.

When the number of cross roads per mile exceeded 2, average overall speeds increased slightly with an increase in driveway frequency from less than 10 to 10–20 per mile. This slight increase reflects the circumstance that in mountainous terrain, where there are likely to be few farm entrances, road grades adversely affect vehicle speeds; while in flat or rolling terrain, where there is usually a greater frequency of farms, road grades are more conducive to higher overall speeds. It will be noted that average overall speed dropped abruptly for a driveway frequency in excess of 20 per mile.

When there were more than 2 cross roads per mile, no change in rate of fuel consumption resulted for an increase in driveway frequency from less than 10 per mile to 10–20 per mile. However, an increase in driveway frequency to over 20 per mile resulted in an appreciable reduction in fuel consumption, presumably due to the decreased speeds brought about by the increased frequency of driveways.

Traffic signals, access points, and sharp curves

Certain of the effects of traffic signals, access points, and sharp curves on passenger-car operation and the frequency of occurrence of these impedances were computed, using data collected with the traffic impedance analyzer. These are presented in table 4, differentiated according to whether the impedances were in rural or urban areas.

One important effect of a traffic impedance is to cause changes in vehicle speeds. The average number of speed change units shown in table 4 for each of the three impedances is the average for each impedance of the arithmetic sum of all speed changes associated with the movement of a vehicle past the impedance, each speed change unit being a change in speed of 1 m.p.h. For example, if a vehicle approaching a traffic signal at 50 m.p.h. slows to 25 m.p.h., increases speed to 30 m.p.h., and then slows to a stop followed later by an increase in speed back to 50 m.p.h., the total number of speed change units would be $(50-25)+(30-25)+(30-0)+(50-0)=110$ units of speed change. All single speed changes of 3 m.p.h. or less were ignored.

The percentage of traffic signals at which the study vehicle was stopped and the average duration of traffic signal stops, as well as the percentage of commercial and residential driveways at which the study vehicle was slowed by vehicles entering or leaving the traffic stream, are included in table 4 as important impedance effects for consideration in benefit studies.

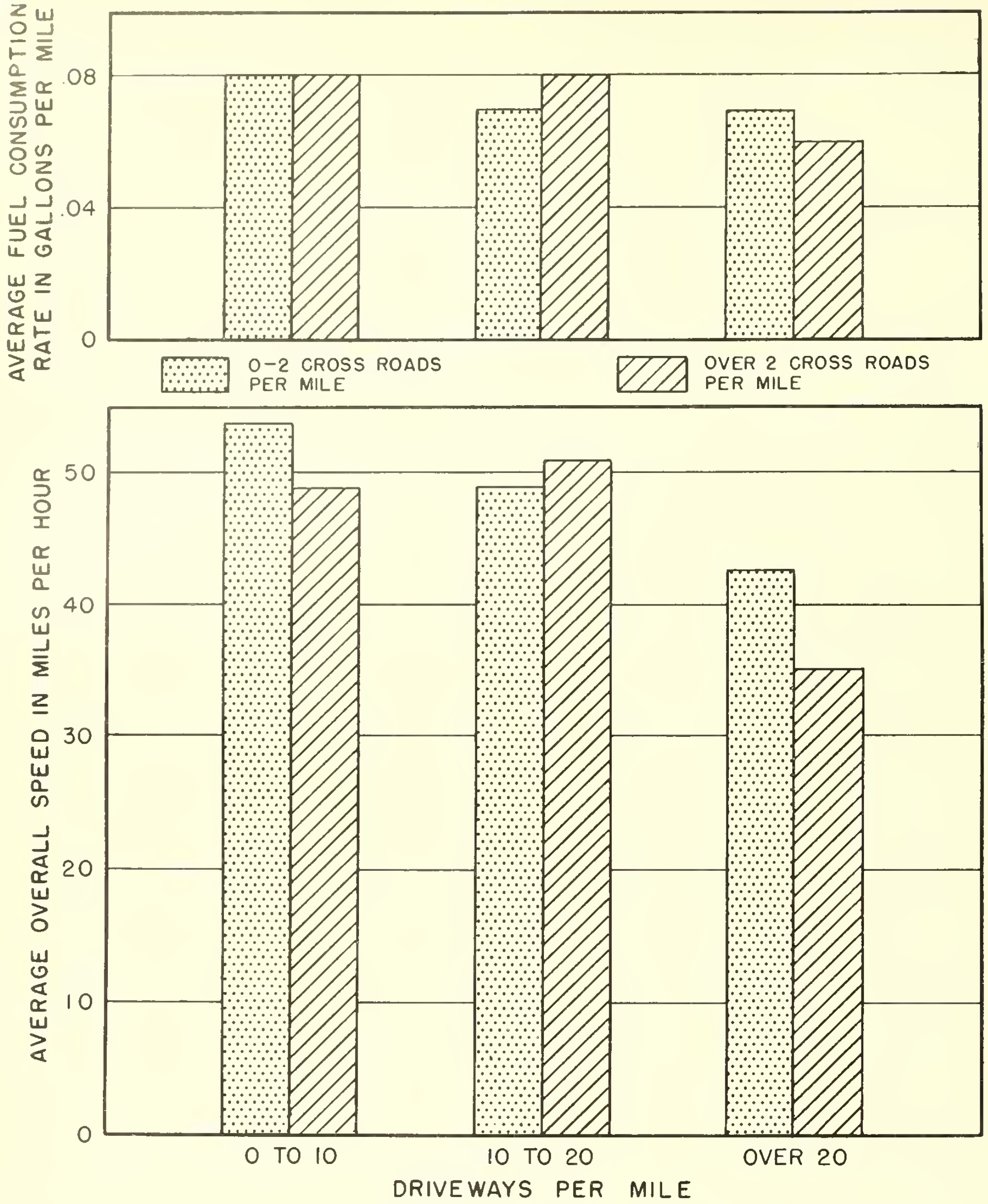

Figure 3.—Average overall speeds and fuel consumption rates on primary 2-lane rural roads as affected by frequency of driveways and cross roads.

Table 4.—Frequency on primary highways of traffic signals, access points, and sharp curves, and their effect on operation of the study vehicle

	Urban areas	Rural areas
Average number of impedances per mile:		
Traffic signals	1.96	[1] 0.05 [1] .20
Residential driveways	16.0	5.8
Commercial driveways [2]	34.2	5.2
All driveways [2]	50.2	11.0
Cross roads	10.6	1.9
Effects of impedances on study vehicle:		
Traffic signals at which study vehicle was stopped...percent	43	30
Average stopped delay at traffic signals...minutes	.29	.21
Average number of speed change units [3] per traffic signal stop	60	90
Driveways [2] at which study vehicle was slowed by entering or leaving vehicle...percent	.5	.8
Average number of speed change units [2] per driveway [3] at which study vehicle was slowed by entering or leaving vehicle	20	20
Average number of speed change units [3] per sharp curve	15	15

[1] 0.05 for 2-lane roads; 0.20 for 4-lane roads.

[2] For wide commercial entrances, each 40 feet of width was counted as one access point.

[3] The number of speed change units for any impedance is the arithmetic sum of speed changes associated with the impedance.

The study vehicle was stopped by 43 percent of the traffic signals encountered in urban areas and by 30 percent of those in rural areas, with an average stopped delay of 0.29 minute and 0.21 minute in urban and rural areas, respectively.

The percentage of driveways at which the study vehicle was slowed by vehicles entering or leaving the highway at driveways was 0.5 percent (1 in 200) in urban areas and 0.8 percent (1 in 125) in rural areas. The average number of driveway access points per mile on the free routes studied was 50.2 in urban areas and 11.0 in rural areas. The average distances traveled by the study vehicle between driveways at which an entering or leaving vehicle caused a slowdown were computed, using these values, and were found to be 4 miles in urban areas and 11 miles in rural areas.

The amount of saving in fuel and time consumption that would accrue to users through the elimination of slowdowns due to driveway entrances and stops due to traffic signals can be computed using these data together with data on the fuel and time consumed by highway vehicles for stop-and-go and slowdown operations (*1*).

The usefulness to benefit studies of the speed change data shown in table 4 will be described in a later section of this article, in connection with the value to users of improved driving comfort.

Relative Use of Toll and Alternate Free Routes

Roadside interviews were conducted at stations on both the toll road and the alternate free route at each toll route comparison section on which the study vehicle was operated, to obtain information both on the relative use of the two routes and on the factors affecting user selection of one route in preference to the other. The drivers of all passenger cars operating in the direction from the initial point to end point of each comparison section during a 1-day interview period (8 a.m.–8 p.m.) were stopped at each interview station and asked the following questions:

1. What is the origin of this trip? (If the origin reported was the city in which the initial point of the comparison section was located, the driver was asked to give the street address.)
2. What is the destination of this trip?
3. What is the purpose of this trip?
4. Why are you using this route rather than the alternate toll/free route?

The two interview stations for each comparison section were operated by the highway department of the State in which the particular section was located, during the week that the test runs were being made.

The data obtained at the roadside interview stations at both the toll and free routes of each of the 14 toll route comparison sections were analyzed to determine the relative use of high-type roads on which a toll is levied and alternate free routes of lower design standards. Since the interview data included trip purposes and driver responses to the interview question regarding drivers' reasons for selecting whichever route they were interviewed on, it was possible to extend the analysis to show the relative importance of some of the factors influencing driver choice of route.

In computing the percentage of the drivers electing to use the toll road at each comparison section (as given in table 6), only drivers going the full length of the comparison section whose trips originated at the initial point of the particular section were included. Drivers whose trips originated beyond the origin city

Table 5.—Drivers' reasons for choice of toll road or comparable free road, by route and by purpose of trip [1]

Section No. or purpose	Number of drivers responding	Percentage of drivers using selected route for—				
		Time saving	Greater safety	Less costly	Greater comfort and convenience	All other reasons
DRIVERS ON TOLL ROADS, BY ROUTE SECTION						
1	94	66	3	1	18	12
18	65	83	2	0	9	6
19	321	71	5	0	20	4
2	281	84	5	0	4	7
3	84	71	4	0	21	4
4	3,260	82	1	(2)	11	6
51	689	78	3	0	15	4
6	4,214	81	5	2	12	0
7	154	79	5	0	8	8
11	126	76	7	0	14	3
DRIVERS ON TOLL AND FREE ROADS, BY TRIP PURPOSE						
Work:						
Toll	2,440	83	3	2	5	7
Free	2,995	25	0	16	20	39
Shop:						
Toll	118	83	4	1	6	6
Free	590	31	0	9	23	37
Vacation:						
Toll	4,784	81	4	(2)	6	9
Free	2,864	12	(2)	10	15	63
Other social or recreation:						
Toll	1,286	78	3	(2)	11	8
Free	1,497	24	(2)	11	24	41
Other:						
Toll	660	73	4	1	9	13
Free	627	32	0	14	19	35
Total:						
Toll	9,288	80	4	1	7	8
Free	8,573	21	(2)	13	19	47
Grand total	17,861	52	2	6	13	27

[1] See table 2 for routes and terminal points of comparison sections. Sections 8, 9, 10, and 20 are omitted because data analysis was incomplete.

[2] Percentage negligible.

were excluded because it was felt that these drivers were not local people and not sufficiently aware of the travel characteristics of the comparable routes to make a rational choice. Drivers whose trips originated or ended at intermediate points were excluded since, in most cases, they would have to go an appreciable distance out of their way to make use of the alternate route.

Two significant items of information useful in passenger-car user benefit studies were brought out by the interview study: The relative importance of the factors inducing passenger-car users to elect to use a free road when a toll route is available; and the effect of trip purpose on the relative importance to passenger-car users of the factors inducing them to use either toll routes or free routes.

Reasons for choice of route

The upper portion of table 5 shows for 10 of the toll route comparison sections the breakdown of the toll road user responses to the interview inquiry as to why they elected to use the toll road instead of the free road. The reasons given by all drivers responding to this inquiry were included, whether or not their trip originated at the initial point of the comparison.

The breakdown of avowed reasons for using toll roads shows that between 66 and 84 percent of the toll road users elected to use the toll route rather than the free route to save time, 1 to 7 percent for reasons of safety, 2 percent or less to save money, up to 21 percent for improved driving comfort and convenience, and up to 12 percent for other reasons. Time saving was obviously the most important single factor inducing drivers to travel on toll roads, with improved driving comfort and convenience second in importance. Greater safety appeared to be a relatively modest factor; and cost was negligible, as would be expected. These data indicate that from the passenger-car users' point of view the highway improvements which bring about the greatest benefits are those which reduce time consumption and improve driving comfort and convenience.

The lower portion of table 5 shows for the same 10 toll routes and free routes the analysis of reasons given by the drivers for electing to use the route they were interviewed on, for each of five categories of trip purpose: Work, shop, vacation, social or recreation other than vacation, and all others.

Considering the total responses, an average of 21 percent of free road users indicated that they decided to travel on the free road to save time, 19 percent made the choice for greater driving comfort and convenience, 13 percent to save money, and 47 percent for other reasons. Only a negligible number of the free road users thought travel by the free route was safer. It will be noted that the most common reasons given by free road users for using the free route were those in the category of "all others," which included less driving monotony, desire to shop or visit at points on the free route, see a particular view, and mechanical difficulty with the vehicle. It appears that on the free roads, as on the toll roads, when only factors associated with the highway itself are considered, time saving and improved driving comfort and convenience are of the greatest importance to the passenger-car user. The percentage of passenger-car users that used the free route for reasons of greater comfort and convenience was appreciably higher than the percentage of toll road users giving this reason, which may be explained by the greater frequency and wider choice of restaurants, motels, and gasoline service stations on the free routes.

Considering individual trip purposes, the toll road users indicated that the relative importance of the various reasons for using the toll road was about the same regardless of trip purpose, except that the toll road users traveling to or from work or on shopping excursions were influenced somewhat more by time saving and less by comfort and convenience than were other users. For free road users the relative importance of the reasons for using the free route were also nearly the same for all trip purposes except that most users on vacation used the free road for "other" reasons. Only a relatively small percentage of free road users on vacation used the free road either to save time or for greater driving comfort. In general these data indicate that there is no appreciable overall difference in the importance to users of the various types of benefit by trip purpose.

Motorist's Evaluation of Time Saving and Increased Driving Comfort

The data collected at the roadside interview stations and by operation of the study vehicle on the toll roads and alternate free routes at the toll route comparison sections were analyzed to obtain estimates of the value to motorists of the time saving and the greater driving comfort and convenience they expected or experienced when operating on the toll road. Drivers do not, in general, consciously assign a separate value to each of these benefits. However, since each is effective in influencing driver selection of route, each has a certain amount of attractiveness to users, which may be measured in monetary terms.

The analysis was based upon the theory that through travelers using toll roads, where a free alternate route is available, pay a premium to do so because they expect to benefit by an amount at least equal to the toll charge. The benefits received would be one or more of the following types of benefit: Reduced operating costs, reduced accident expectancy and costs, time saving, and increased driving comfort and convenience. The value of the first two of these benefits can be estimated: Operating cost saving, on the basis of the fuel consumption difference on the two routes; accident cost saving, on the basis of published accident rate and accident cost reports. The problem is to arrive at a value of the other two benefits—time saving and increased driver comfort—on the basis of estimated values of the first two benefits, and a known toll charge presumably paid to obtain these benefits.

Table 6.—Relative use of toll roads and comparable free roads; trip lengths, free road lanes, and toll road toll charges; and reduction in accident cost expectancy on toll roads

Section No. [1]	Passenger cars originating at initial point			Trip length		No. of traffic lanes on free road	Toll charge on toll road	Reduction in accident cost expectancy on toll road [2]
	Using toll road	Using free road	Percent using toll road	Toll road	Free road			
			P				*R*	ΔA
			Pct.	*Mi.*	*Mi.*		*Cents*	*Cents*
1	9	5	64	103.8	107.2	4	130	11.0
18	65	15	81	55.8	58.7	4	60	6.1
19	234	883	21	29.2	29.4	4	30	3.0
2	199	368	35	53.5	50.4	2	75	4.8
3	51	28	65	240.1	216.7	2	370	19.5
4	317	159	67	16.0	16.1	2	20	1.6
51	241	155	60	49.8	48.3	2	100	4.7
6	150	142	51	68.9	67.8	2	90	6.7
7	31	9	78	69.3	62.9	2	85	5.7
8	210	110	66	27.2	26.2	2	30	2.5
9	112	14	90	134.8	165.6	2	245	19.0
10	512	72	88	86.2	98.6	2	140	10.8
20	144	31	82	71.3	80.3	2	120	8.4
11	63	185	25	65.0	57.4	4	100	5.1
Total	2,338	2,176	52	------	------	------	------	----------

[1] See table 2 for routes and terminal points of comparison sections.

[2] Based on unit accident cost expectancy of 0.07 cent per vehicle-mile on toll routes and 0.17 cent on routes without access control.

A difficulty which complicated the problem of evaluating time saving and driver comfort benefits was selection of a suitable unit with which to measure driving comfort. A minute of time could be used to measure time saving but there was no similar unit for measuring improvement of driving comfort. However, it is generally recognized that uniformity of driving speed is a characteristic of good driving conditions. Most of the highway factors that cause driver annoyance such as traffic lights and sharp curves cause vehicles to change speed, frequently causing them to reduce speed to a full stop. These considerations lead to the selection of the speed change unit of 1 m.p.h., already mentioned in connection with table 4, as the unit of driving discomfort. Each speed change unit eliminated through highway improvement is therefore considered a unit of driving comfort improvement. In determining the number of units of speed change for a highway, only variations in speed where the speed change was more than 3 m.p.h. were included since variations up to about this magnitude are typical of normal driving under the best of conditions. Using speed change units as a measure of driving discomfort, the driving discomfort of a section of highway is the arithmetic sum of all speed changes on that section of road, neglecting all single speed changes of 3 m.p.h. or less. The unit value of improved driving comfort is taken in this analysis to be the value to users of each speed change unit of 1 m.p.h. saved through highway improvement.

The data obtained at each of the 14 toll route comparison sections are summarized for the analysis of the motorist's evaluation of time saving and increased driving comfort in tables 6 and 7. Since at each comparison section a series of trips were made on both the free route and toll route, all cost data are given in these tables as differences between the values for a free route trip and a toll route trip. For this purpose each free route trip was paired with a toll route trip and the value of the differences in time consumption, speed change units, and fuel cost, as well as total cost differences, are shown for these comparison trip pairs in table 7. For each comparison section there were two or three comparison trip pairs, identified by the letters A, B, and C.

Accident cost expectancy

In table 6 are presented data on the percentage of drivers electing to use the toll route, the trip lengths, both of the toll and free routes, the number of traffic lanes on the free routes, the toll charge on the toll roads, and the reduction in accident cost expectancy on the toll roads as compared with the free routes. The percentage of drivers electing to travel by toll route was determined from the interviews, using only the drivers on the compared routes whose trip origins were at the initial points of the compared sections.

The saving in accident cost expectancy for operation on the toll route as shown in table 6 is the difference in the average cost of accidents for a passenger car traversing the full length of each comparison section over the toll route as compared to operation over the free route. The accident expectancy cost of a passenger car on each route is the product of the route length in miles, the average accident rate of all types of accidents per vehicle-mile, and the average cost of a passenger-car accident. The average accident rates used in these computations were 151 accidents per 100 million vehicle-miles on toll roads and 332 accidents per 100 million vehicle-miles on roads with no access control, as reported in *The Federal Role in Highway Safety* (*5*).

The average cost of a passenger-car accident on primary rural routes with no control of access was determined as $521 per accident by a study of accident costs in Massachusetts in 1953 (*6*). Certain types of accidents that occur on such highways very seldom happen on divided controlled-access highways (such as the toll roads), including head-on collisions, head-on side-swipe collisions, and collisions with bicycles and scooters. While separate data on such accidents were not available for primary rural routes alone, unpublished information from the Massachusetts accident study showed that for primary and secondary rural highways combined the average cost of such accidents amounted to $33. Using this value as the saving in average passenger-car accident cost achieved through the elimination of the accident types mentioned as unlikely to occur on toll roads, the average cost of an accident on toll routes is derived as $521−$33=$488.

Cost differences

The cost differences for operating over the routes of each comparison section are given in table 7 for each comparison trip pair. The total additional cost for operation on the toll route as compared to operation on the free

Table 7.—Cost differences for operation on toll road as compared to alternate free road

Section No. and trip pairs [1]	Savings or additional costs for operating on toll road as compared to free road [2]			
	Saving in time	Saving in speed change units	Additional fuel cost	Total additional costs [3]
	ΔT	ΔD	ΔF	ΔM
Sec. 1:	*Min.*		*Cents*	*Cents*
A	61	3,410	−1.5	117.5
B	47	3,430	6.9	125.9
C	69	4,290	.9	119.9
Sec. 18:				
A	34	1,795	−6.9	47.0
B	30	2,140	1.2	55.1
C	28	2,055	1.5	55.4
Sec. 19:				
A	13	595	−2.7	24.3
B	7	355	1.8	28.8
C	5	420	2.7	29.7
Sec. 2:				
A	15	945	11.7	81.9
B	16	1,540	8.4	78.6
C	28	2,020	15.3	85.5
Sec. 3:				
A	62	4,580	105.3	455.8
B	60	4,970	99.0	449.5
Sec. 4:				
A	4	110	1.8	20.2
B	7	420	5.1	23.5
C	5	145	2.4	20.8
Sec. 51:				
A	26	1,915	21.6	116.9
B	22	1,190	16.5	111.8
C	31	1,280	18.0	113.3
Sec. 6:				
A	11	410	3.6	86.9
B	9	310	12.6	95.9
Sec. 7:				
A	0	975	38.4	117.7
B	2	865	33.0	112.3
C	3	745	45.0	124.3
Sec. 8:				
A	5	50	14.7	42.2
B	5	35	12.9	40.4
C	4	50	6.0	33.5
Sec. 9:				
A	53	1,390	50.1	276.1
B	51	945	30.6	256.6
Sec. 10:				
A	38	2,040	−7.5	121.7
B	35	1,960	−12.3	116.9
C	39	2,475	−1.5	127.7
Sec. 20:				
A	19	915	24.6	136.2
B	16	705	19.8	131.4
C	23	830	23.4	135.0
Sec. 11:				
A	2	425	52.5	147.4
B	0	475	52.2	147.1

[1] See table 2 for routes and terminal points of comparison sections. A comparison trip pair consists of one toll road trip and one free road trip.

[2] Savings in time and speed change units and additional fuel cost are for full trip length.

[3] Total additional costs $\Delta M = R + \Delta F - \Delta A$. Values for R and ΔA appear in table 6, and are identical for all trip pairs on a given route section.

route, for each comparison pair, is found by use of the formula:

$$\Delta M = R - \Delta A + \Delta F,$$

where ΔM is the additional cost of operation on the toll road, R is the toll charge on the toll road, ΔA is the reduction in accident cost expectancy for operation on the toll road (table 6), and ΔF is the additional fuel cost for operation on the toll road. The values for savings in time and speed change units, and the additional fuel costs, all reported in table 7, were derived from the data collected by the study vehicle.

Unit time and speed change values

The motorists at the initial points of each comparison section who wish to travel the full distance from the initial point to the end point of a comparison section are faced with the choice between two alternatives: (1) To use the toll road and pay the extra cost ΔM but save an amount of time, ΔT, and a number of speed change units (driving comfort), ΔD; or (2) to use the free road and put up with the additional time consumption and speed changes but save the total cost difference, ΔM. For the analysis, the percentage of drivers who elected to use the toll road at each location is represented by P, as given in table 6.

The first step undertaken in the analysis of the data to obtain estimates of t, the value the average motorist places on the saving of one minute of trip time, and s, the value of saving one unit of speed change, was the establishment of a relationship between these unknowns and the study data, ΔM, ΔT, ΔD, and r, where:

$$r = P/(100-P).$$

Equations defining a model of the relationship between these variables were presented by Nathan Cherniack at a workshop conference on economic analysis held in Washington, D.C., in September 1959 (*7*). The following simple equation is an adaptation of Cherniack's work appropriate to this analysis as developed by G. P. St. Clair, Director of the Bureau of Public Roads Highway Cost Allocation Study:

$$+\Delta M = -u \log r - t\Delta T - s\Delta D.$$

Mr. St. Clair's derivation of this equation is presented as an appendix to this report (see below).

The values of t and s were arrived at by substituting the values of ΔM, ΔT, ΔD, and r (the latter being derived from $P/100-P$) from tables 6 and 7, in a series of equations of the above form and solving by multiple regression. Only data for runs at study locations where the free routes are 2-lane roads were included in the equations. It was considered best not to include data for both 2-lane and 4-lane free roads in the equation for one multiple regression solution because of the differences in travel characteristics on the two types of road. Of particular concern was the fact that passing maneuvers measured by the amount of speed change represent a greater annoyance to drivers on 2-lane roads than they do on 4-lane roads. The data used in the multiple regression solution are those determined for the 27 comparison trip pairs of the 10 comparison sections where the free routes are 2-lane roads (see table 6).

The computations of the values of t and s by multiple regression analysis were made by Nathan Lieder, statistician for the Office of Research of the Bureau of Public Roads. The values of t and s, together with the confidence limits on the 95-percent level of accuracy, were found to be the following:

$t=2.365$ cents per minute ± 0.059 cent.

$s=0.048$ cent per speed change unit ± 0.062 cent.

The estimate of the motorist's evaluation of a minute of time saved is thus 2.37 cents on the basis of the data collected for this study, which agrees fairly well with the estimate recommended by the American Association of State Highway Officials, 2.58 cents per minute (*3*).

The estimate of the value to motorists of the elimination of one speed change unit (a 1 m.p.h. change in speed) is 0.05 cent. The variance of this estimate, ±0.06 cent, is very high, however, and appears to indicate that driver discomfort is not fully measured by speed change units. Certain anomalies in the data and general observations of the field crew of this study also indicate that driver discomfort is greater than shown by the number of speed change units. For example, one obviously annoying traffic condition is for a motorist to have to trail a slow-moving vehicle for many miles on a 2-lane road before finding an opportunity to pass. The trailing driver, forced to travel at a slow but uniform speed, is annoyed because his speed is controlled by another driver and because he knows that to gain relief he must pass on a 2-lane road, which in itself is annoying. However, this distress is not reflected in speed change units.

It is evident, therefore, that further investigation is needed: First, to obtain more data on speed changes on route comparison sections; and second, to incorporate into the analysis of the motorist's evaluation of relief from driving annoyance, other measures (of annoyance) in addition to speed change units. In connection with the latter, further analysis is planned to exploit field data on the trailing operations of the study vehicle on 2-lane roads which were collected for this study but not contained in this report.

The value to passenger-car users of a minute of time saving (2.37 cents) is considered accurate since the average number of units of time saving (minutes) can be directly measured. The analysis thus gives an accurate distribution between the items of time saving and reduced driving annoyances of the average passenger-car user's evaluation of the sum of these two benefits. The relatively high value arrived at for each unit of speed change saving (0.048 cent) is due to the allocation of the travel discomfort benefit value among only the number of speed change units saved, when it probably should be allocated among the number of speed change units saved plus an unknown number of other discomfort units saved. To the extent that the amount of driving discomfort not measured by number of speed change units is in any way related to the number of speed change units, the product of 0.048 cent and the number of speed change units saved through highway improvement is a reasonable estimate of the value of the improved driving comfort benefit arising through the improvement. This estimate of the user's evaluation of each 1 m.p.h. speed reduction may be used to approximate driving comfort benefits arising through improvements which reduce the number of speed change units on roads similar to those for which study data were obtained, primary rural roads without control of access.

Impedance elimination values

In table 4 the average numbers of speed change units associated with three impedances—a traffic signal stop, an access point where a through vehicle is slowed by an entering or leaving vehicle, and a sharp curve—are shown for operation in rural areas. Multiplying these values by the estimate of the average motorist's evaluation of the elimination of one speed change unit, 0.048 cent, gives the following estimates of the comfort and convenience benefits users receive through the given highway improvements:

Elimination of a traffic signal stop in rural areas	4.32 cents.
Elimination of a sharp curve	0.72 cent.
Elimination of a slowdown (for a through vehicle) at an access point	0.96 cent.

APPENDIX

The following is a derivation of a simple equation relating time saving, saving in speed change units, and cost difference for operation on a toll road rather than a free road, and the percentage of traffic using the toll road, to user evaluations of a minute of time saving and a 1 m.p.h. reduction in amount of speed change.

Assume that the number of daily round trips that will be made from a given origin O, to a given destination D, for a given purpose, can be represented by the following equation:

$$V_{OD}=q\times V_O\times I_{Da}\times F(C) \qquad (1)$$

where

q=a coefficient related to the propensity to make trips. This value is immaterial for present purposes.
V_O=number of vehicles domiciled in zone of origin.
I_{Da}=coefficient of attractiveness of the destination zone D for trips of the given purpose a. Then, if the purpose is home-to-work, this coefficient would be related to the number of employed persons in zone D.
$F(C)$=a function of the average cost of trip, C.

The number of trips for all purposes is given by the equation:

$$V_{OD}=q\times V_O\times F(C)\times \Sigma I_n$$

where $\Sigma I_n=I_a+I_b+I_c+\;.\;.\;.\;.\;I_n$, the sum of all trip purpose coefficients.

Let the following represent the cost function:

$$F(C)=10^{-hC} \qquad (2)$$

where h is a coefficient to account for the unknown effect of travel cost on driver trip decisions.

This equation can be simplified by putting $h=1$, but this presumably would reduce its generality and force it to conform to a curve that the data might not fit. Furthermore, $h=1$ lacks generality, since if base e were used, it would produce a different function.

In general this function behaves more or less as would be wished. If C is limited to positive values, as it should be, the function has its greatest value, one, when $C=0$; and thus the number of trips varies inversely with the cost. It is not a perfect function but it has the virtue of simplicity.

The equation therefore takes the form:

$$V_{OD}=q\times V_O\times \Sigma I_n\times 10^{-hC} \qquad (3)$$

Alternative routes.—Equation 3 may be taken as applicable to all trips made from O to D, the cost C being taken as the average cost of the trip OD. There is, however, the problem of the distribution of the trips V_{OD} among two or more alternative routes having different trip costs. The equation will presumably hold for any one alternative, in relation to numbers of trips to *other destinations*. To assume that it holds for the distribution of trips to the same destination among alternative routes, it is necessary to say that the distribution of motorists' subjective appraisals of certain cost elements (values of time and driving comfort) is such that the distribution of trips among alternative routes between the same termini obeys the same cost function.

If we reduce the subscript OD to O or D, according to the point of origin, and take the suffix numbers 1 and 2 for two alternative routes between O and D, we may write, for trips from O to D:

$$V_{O1}=q\times V_O\times \Sigma I_D\times 10^{-hC1}$$

$$V_{O2}=q\times V_O\times \Sigma I_D\times 10^{-hC2}$$

$$r_O=\frac{V_{O1}}{V_{O2}}=\frac{10^{-hC1}}{10^{-hC2}}=10^{-h(C1-C2)}$$

It becomes at once obvious that the result would be exactly the same for trips originating at D:

$$r=r_O=r_D=10^{-h(C1-C2)}$$

$$\log r=-h(C1-C2)$$

$$\log r=-h(\Delta C)$$

$$\log r=-h(\Delta M+t\Delta T+s\Delta D) \qquad (4)$$

where:

ΔM=the net sum of measured cost differences: operating cost, accident cost expectancy, and toll charge.
ΔT=time difference.
t=unit value of a minute of time saved.
ΔD=difference in speed change units.
s=unit value of a speed change unit eliminated.
$r=P/(100-P)$
P=percentage of sum of travelers on the two alternate routes who elect to use the toll road.

The signs of the terms must be watched. If the free route is designated as route 1, then ΔM is likely to be negative and ΔT and ΔD are likely to be positive because measured costs are less on the free route, while time and driving comfort costs are generally greater.

Taking $u=1/h$, then, from equation (4):

$$u\log r=-\Delta M-t\Delta T-s\Delta D;\text{ and}$$

$$\Delta M=-u\log r-t\Delta T-s\Delta D \qquad (5$$

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Moisture Content Determination by the Calcium Carbide Gas Pressure Method

BY THE DIVISION OF PHYSICAL RESEARCH
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A rapid, reliable method of determining soil moisture content in the field with a simple, portable apparatus is an important need in highway engineering. A commercially made device for this purpose, which shows great promise, measures moisture content indirectly by gaging the pressure of gas generated when calcium carbide reacts with the moisture in a small soil sample. Tests of the apparatus are described in this article, which includes calibration curves for relating the gage readings to moisture content derived by the oven-drying method. Test results varied with the time of mixing of the calcium carbide and soil, and a 3-minute mixing time is recommended. Time-temperature relations of the chemical reaction were observed, but no significance in this relation with respect to moisture content was found.

Introduction

A QUICK AND ACCURATE METHOD of determining moisture contents in soil materials has long been one of the goals of soil engineers. Researchers have devoted much effort in developing gravimetric, chemical, electrical, nuclear, penetrometer, tension, and thermal methods for the determination of the moisture content of soils.[2] A chemical method using calcium carbide as a reagent is showing great promise.

The principle involved in this chemical method for moisture determination is that a given quantity of moisture will react with calcium carbide to produce a specific volume of gas (acetylene). The reaction is as follows: $CaC_2+2H_2O=Ca(OH)_2+C_2H_2$. Based on this principle, a device was developed in England that confined in a pressure vessel the gas produced from this reaction. The gas pressure is read on a gage located on one end of the pressure vessel, the gage being calibrated to read in percentage of moisture based on the wet weight of the sample.

Although the device has been used in England for several years for moisture determinations of foundry sand, it has only been used by highway engineers in the United States for about 2 years. This article describes the results of a study by the Bureau of Public Roads to determine the accuracy and usefulness in highway soil testing of a commercially manufactured "moisture tester."

Apparatus and Testing Procedure

The moisture tester is a hollow aluminum vessel having a pressure gage on one end and a cap with a clamping arrangement on the other. The manufacturer makes two sizes of moisture testers, one for a 6-gram sample and the other for a 26-gram sample. Only the larger device, having an approximate length of 14 inches, a cylinder diameter of 6 inches, and a weight of 3.7 pounds, was used in this study. This moisture tester is shown in figure 1. The tester with its auxiliary equipment—a carrying case, a tared scale for weighing the sample to be tested, a small scoop for measuring the calcium carbide, and a table to convert the moisture percentage to the dry weight basis used in soils work—is illustrated in figure 2.

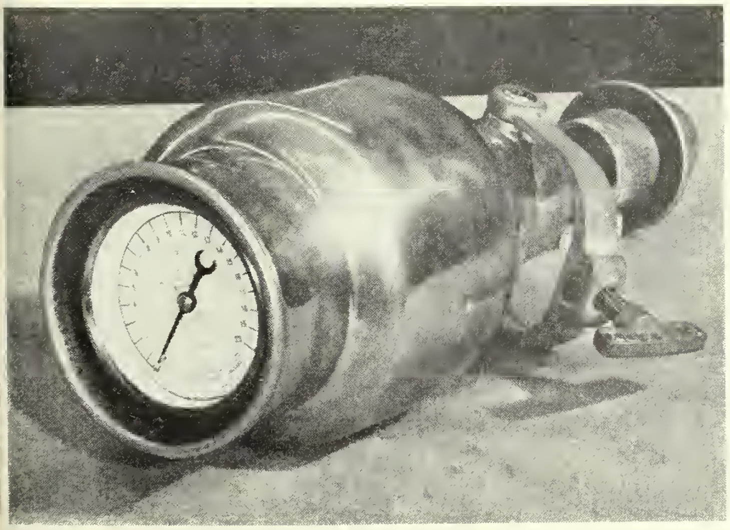

Figure 1.—Moisture tester.

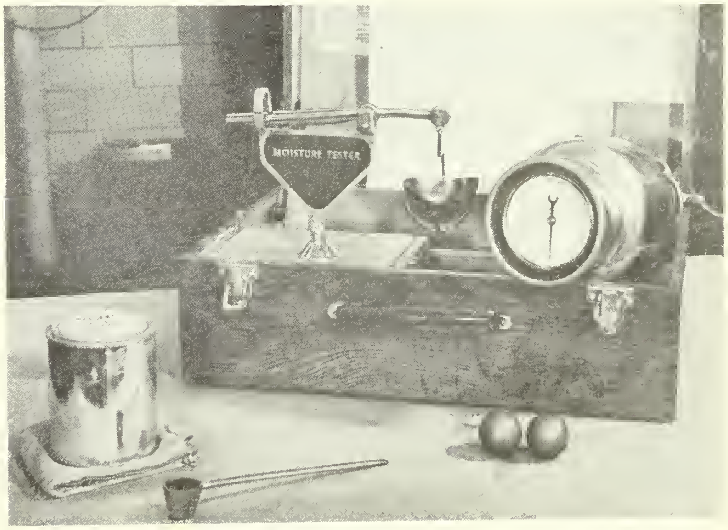

Figure 2.—Moisture tester with accessories.

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[1] Presented at the 40th annual meeting of the Highway Research Board, Washington, D.C., January 1961.

[2] See *Bibliography on Methods for Determining Soil Moisture*, by M. D. Shaw and W. C. Arble, Pennsylvania State University, Engineering Research Bulletin B-78, 1959.

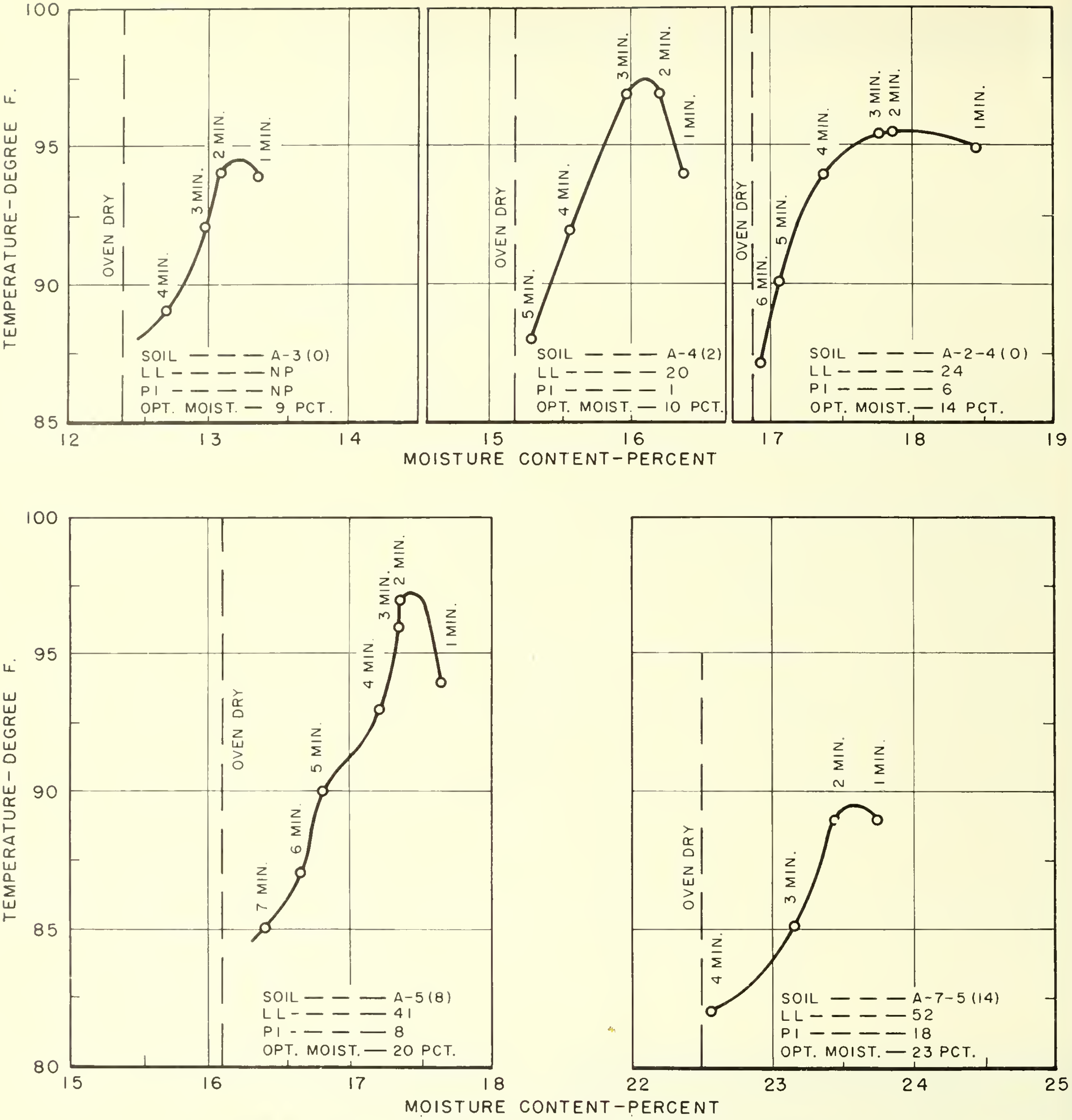

Figure 3.—Moisture content determined by moisture tester as related to temperature.

Preliminary tests using the procedure recommended by the manufacturer indicated that some changes in method were desirable. Consequently, a variety of soil samples passing the No. 4 sieve, and ranging from group A–2 through group A–7, were used in a procedure study.

The soil samples were tested at varying moisture contents in the moisture tester and by the standard oven-dry procedure. To arrive at a procedure that would give moisture contents as close as possible to those determined by oven drying, tests were performed using varying combinations of shaking time, amount of reagent, and types (shape, size, and weight) of pulverizers to break up clay lumps. The most effective pulverization of clay lumps was obtained by placing two 1¼-inch steel balls, weighing 0.4 pound each, in the moisture tester with the soil sample and calcium carbide, and agitating the device. The procedure adopted for use of the moisture tester as a result of these trials is outlined in the appendix, page 181.

Temperature Study

The chemical reaction of the calcium carbide with the moisture in some soils caused a rapid rise in temperature in the moisture tester, and it was decided to study this thermal effect more fully. An experimental program was undertaken to determine if a correlation could be established between temperature and moisture tester readings. Eleven soils, ranging from group A–2–4(0) to group A–7–5(20),

Table 1.—Physical characteristics of soils used in temperature study

AASHO soil classification	Liquid limit	Plasticity index	Optimum moisture content [1]	Range of moisture contents tested [2]
			Percent	*Percent*
A-2-4(0)	24	6	[4] 14	8.8-16.9
A-3(0)	NP [3]	NP [3]	[4] 9	4.3-15.2
A-4(2)	30	6	[4] 17	11.8-22.0
A-4(2)	20	1	10	5.8-15.2
A-4(8)	26	3	15	9.6-19.8
A-4(3)	35	9	15	11.1-22.3
A-5(8)	41	8	20	14.1-25.5
A-7-5(6)	42	11	20	15.1-28.0
A-7-5(14)	52	18	23	18.4-29.1
A-7-5(19)	66	27	27	23.8-34.4
A-7-5(20)	82	47	[4] 33	29.4-41.9

[1] AASHO Designation T 99-57, Method A.

[2] Moisture contents determined by standard oven-dry method.

[3] Nonplastic.

[4] Estimated optimum moisture content.

were selected for these tests. Their characteristics are given in table 1.

A thermometer was taped to the outer surface of the moisture tester at about the midpoint of the cylinder. Although the thermometer was insulated from external temperatures, it was recognized that the temperature recorded was not the actual temperature inside the moisture tester. However, for purposes of determining whether a correlation existed between temperature and moisture tester readings, it was not considered essential to know the actual inside temperature of the moisture tester.

After adding water to the soil, the moisture content was determined by (1) the standard oven-dry method, and (2) the moisture tester, using the procedure described in the appendix except that readings were taken at 1-minute intervals. When a soil was tested in the moisture tester, the initial temperature prior to the mixing of the soil and the calcium carbide was recorded, then the soil and chemical were mixed, and readings of the pressure gage and thermometer were recorded at 1-minute intervals until the temperature returned to within 5° F. of the initial temperature.

In this series of tests, moisture was added to the selected soil in increments of about 2 percentage points until the moisture content was past the optimum moisture content. Two successive tests were performed with the moisture tester for each increment of moisture tested. The readings of such duplicate tests were often identical or within 0.1 percentage point of each other, and most were within 0.5 percentage point of each other. The range of moisture contents tested for each soil is shown in table 1. If the moisture contents of the soils exceeded the limit of the pressure gage, half-sized samples (13 grams) were used and the percentage indicated on the gage was then doubled. There were 1,598 recordings of temperature and moisture tester readings for the selected soils in this temperature study.

The moisture tester readings were converted to a dry weight basis using the converison chart supplied by the manufacturer. For each test, diagrams of moisture content as related to temperature were plotted for 1-minute intervals until the temperature returned to within about 5 degrees of its initial value. The oven-dry moisture content was also plotted as a dashed vertical line on each diagram. Some typical diagrams are shown in figure 3.

The curves all fall to the right of the oven-dry moisture content line, indicating that the moisture contents as determined by the moisture tester were greater than as determined by oven drying. The greatest moisture content values were at the 1-minute moisture tester observations. Those moisture content determinations made from the moisture tester observations at 4, 5, and 6 minutes more nearly approach the oven-dry moisture content of the soil than those at lesser time intervals. It was anticipated that a relation might exist between the peak points of the curves and the oven-dry moisture contents, but no such relation could be determined.

Table 2.—Comparison of moisture contents by moisture tester and oven drying

AASHO soil classification	Moisture content determined by—					Moisture content difference [1]	
	Dial reading on moisture tester		Calibration curves		Oven drying	1 minute	3 minute
	1 minute	3 minute	1 minute	3 minute			
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percentage point*	*Percentage point*
A-2-4(0)	5.0	5.0	5.2	5.1	5.3	0.1	0.2
	10.7	10.5	11.4	11.3	11.2	−.2	−.1
	13.8	13.8	15.1	15.4	15.7	.6	.3
A-4(5)	8.9	8.7	9.5	9.3	9.1	−.4	−.2
	12.5	12.3	13.5	13.6	13.5	.0	−.1
	18.0	17.6	20.5	20.6	20.1	−.4	−.5
A-6(10)	11.8	11.6	12.6	12.6	12.8	.2	.2
	15.2	14.9	16.7	16.8	16.5	−.2	−.3
	19.2	18.7	22.0	22.1	21.4	−.6	−.7
A-7-6(9)	14.4	14.0	15.7	15.6	15.6	−.1	.0
	18.9	18.3	21.6	21.5	20.4	−1.2	−1.1
	22.4	21.9	26.7	26.8	26.7	.0	−.1
A-7-5(3)	19.6	19.4	22.6	23.1	24.5	1.9	1.4
	23.0	22.6	27.7	28.1	28.9	1.2	.8
	27.0	26.4	34.2	34.0	35.0	.8	1.0
A-7-5(20)	22.4	22.2	26.8	27.5	29.0	2.2	1.5
	25.2	24.8	31.2	31.4	32.4	1.2	1.0
	29.6	28.8	38.5	38.0	37.9	−.6	−.1
Average deviation	--------	--------	--------	--------	--------	.7	.5

[1] Deviation of moisture content determined by moisture tester and calibration curve from oven-dry moisture content.

Calibration Curves

The moisture content data from this temperature-moisture test program were used in developing two calibration curves for conversion of moisture tester readings to the equivalent of moisture contents obtained by oven drying. Tests covering a range of moisture contents from 4.3 to 41.9 percent were used in developing these calibration curves, shown in figure 4. The moisture tester readings at 1 minute and 3 minutes are plotted against oven-dry moisture contents. The 1-minute readings were selected for a calibration curve since the first recordings of the pressure gage of the moisture tester were taken at 1 minute. A calibration curve was developed for the 3-minute readings since this time was not considered to be excessive for running tests in the field and was adequate for the reaction of the calcium carbide to occur with moisture in clay soils. Either 13- or 26-gram samples were used in the development of the curves.

In order to establish the validity of the calibration curves and to compare the deviations using the 1- and 3-minute curves, six additional soil samples were selected for moisture testing. Each sample was tested at moisture contents bracketing the optimum (AASHO Designation T 99). The oven-dry moisture content was determined for each moisture increment added to the six soils. Using the moisture tester readings at 1 and 3 minutes and the 1- and 3-minute calibration curves (fig. 4), a predicted oven-dry moisture content was determined. These predicted values, along with the actual oven-dry values, and the deviation between them, are shown in table 2.

The moisture content values obtained from the 3-minute readings tended to be more accurate than those for 1-minute readings. For example, the A-7-5(20) soil had the greatest deviations from true moisture content, but the deviation of corrected moisture-tester values from oven-dry moisture content was 2.2 percentage points for the 1-minute reading and only 1.5 percentage points for the 3-minute reading. The average deviation for the 1-minute corrected readings was 0.7 percentage point, while the average deviation for the 3-minute corrected readings was 0.5 percentage point—only a 0.2 percentage-point difference. Nevertheless, it is recommended that the moisture tester readings be taken at the end of 3 minutes, particularly for clay samples, to ensure complete reaction of the calcium carbide with the moisture, and thus greater accuracy.

Conclusions and Applications

The moisture tester has been in use in the soils laboratory of the Bureau of Public Roads for over a year. It has proven to be a sturdy, dependable, and reasonably accurate instrument, and above all, fast and easy to operate. It is believed that by using the procedure

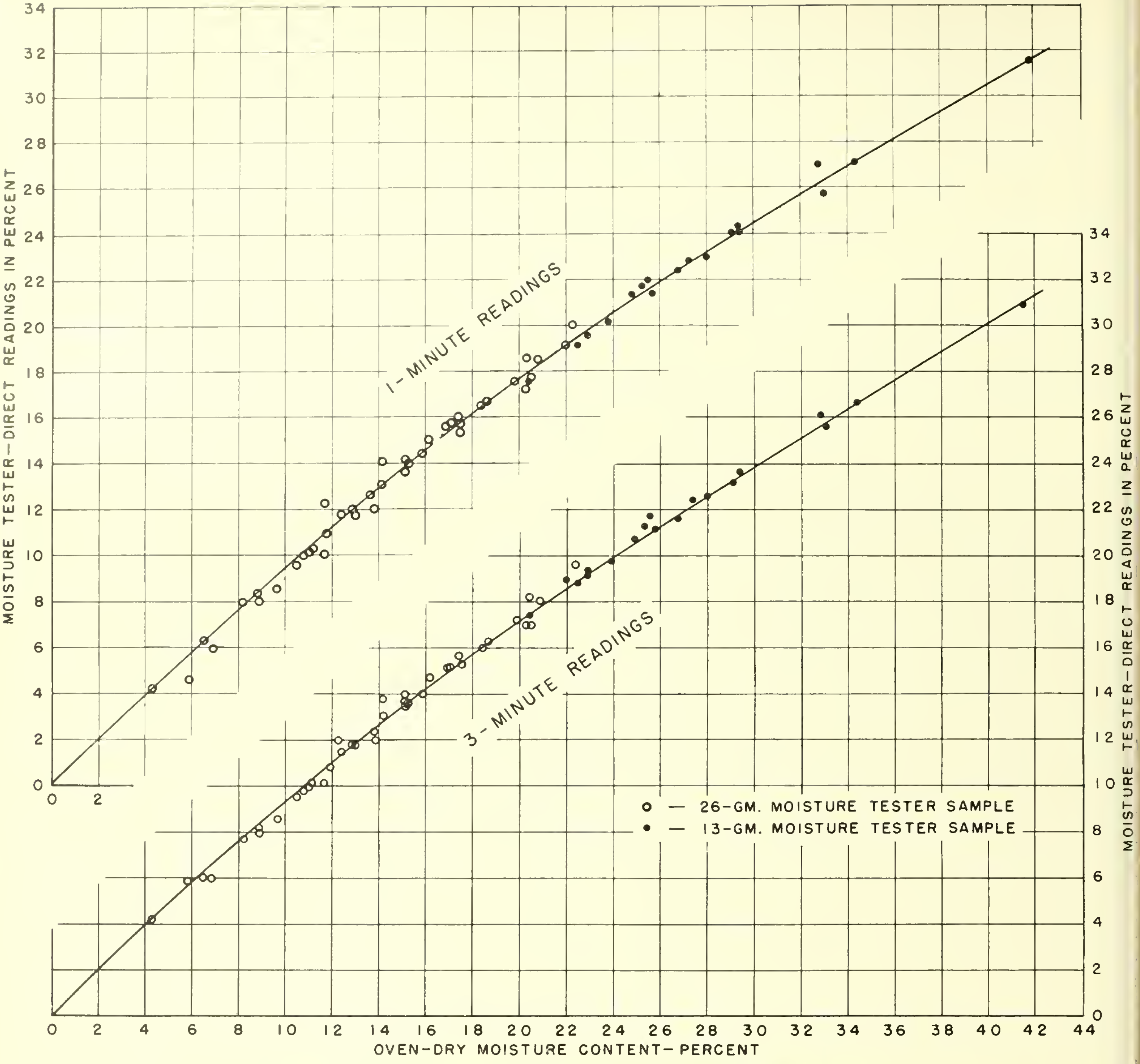

Figure 4.—Calibration curves for 1- and 3-minute moisture tester readings.

outlined in the appendix, even an inexperienced operator will become proficient in a short time. By using the 3-minute calibration curve in figure 4, direct moisture tester readings can be converted to a dry weight basis, and a moisture content value obtained that agrees closely with that obtained by oven drying.

Inasmuch as the calibration curve was developed for the specific moisture tester device tested, it is not known that the curve is applicable to each tester made by the manufacturer. Consequently, a highway department or other agency that purchases this type of moisture tester either should perform check tests with each device to determine that the calibration curve in figure 4 is applicable, or should develop a calibration curve for the specific moisture tester and local soils.

The calibration curve could be used to design a dial with values of moisture content on a dry weight basis, which would expedite moisture content determinations and eliminate possible errors in reading the calibration curve.

There are many applications of the moisture tester in highway engineering. It is well suited for the control of materials and construction practices where reasonable accuracy and rapid operations are required. The moisture tester can also be used as a quick check for field laboratory tests involving moisture content determinations. Among the possible applications of the moisture tester are the following:

Site locations.—In-place density tests; auger and split-spoon samples; sands used in concrete mixtures; and proper moisture content for earthwork.

Field laboratory.—Hygroscopic moisture; development of compaction curves; and low-value liquid and plastic limits.

Thirty-four State highway departments have reported that they have made tests with the calcium-carbide moisture tester and several are using the testers in construction control. This interest by the State highway departments appears to indicate that the calcium-carbide moisture tester is rapidly being accepted by the construction industry and the engineering profession.

Appendix—PROCEDURE FOR USE OF 26-GRAM MOISTURE TESTER [1]

1. Place three measures (approximately 22 grams) of calcium carbide and two 1¼-inch steel balls in the large chamber of the moisture tester.

2. Using the tared scale, weigh a 26-gram sample of soil.[1]

3. Place the soil sample in the cap; then, with the pressure vessel in a horizontal position, insert the cap in the pressure vessel and tighten the clamp to seal the cap to the unit.

4. Raise the moisture tester to a vertical position so the soil in the cap falls into the pressure vessel.

5. Holding the moisture tester horizontally, manually rotate the device for 10 seconds so that the steel balls are put into orbit around the inside circumference, and then rest for 20 seconds. Repeat the shake–rest cycle for a total of 3 minutes. Do not allow the steel balls to fall against either the cap or the orifice leading to the dial, since this might cause damage.

6. Read the pressure gage of the moisture tester and determine the moisture content of the soil on a dry-weight basis from the calibration curve.

---

[1] This procedure is for the 26-gram sized moisture tester. If the moisture content of the 26-gram soil sample exceeds the limit of the pressure gage, a half-sized sample can be used; the percentage indicated on the dial is then doubled. Since the sample to be tested is relatively small (13 or 26 grams), care must be taken to obtain a representative soil sample.

New Publications

Construction Specifications for Federal Highway Projects

The Bureau of Public Roads has recently published a new edition of *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects.* As indicated by the title, the book is intended primarily for use in the construction of Federal road and bridge projects under the direct supervision of the Bureau. Such projects include work for which Public Roads receives direct appropriations, as for major highways through National Forests, and work performed by Public Roads for other Federal agencies.

The new publication, which for simplified reference is identified as FP–61, supersedes the 1957 edition commonly known as FP–57. Numerous changes have been made in an effort to provide up-to-date specifications for those items of work and materials and construction methods that are generally applicable to direct Federal highway contracts. As examples, thicknesses with tolerances are now to be shown for all pavement items; the use of diesel hammers is permitted for pile driving; and the radiograph method of checking welds in steel structures is recognized.

During the revision of the old specifications, the material was reviewed by both field and office engineers of the Bureau of Public Roads and by committees representing national organizations of contractors and of producers and suppliers of materials and equipment. Over a thousand different comments and suggestions were received from these sources. The benefits of these comments and suggestions are reflected in the new book.

The 371-page publication is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at $2.25 per copy.

The FP–61 specifications are not required or intended to be used in Federal-aid highway work performed by the States with funds administered by the Bureau of Public Roads since, as prescribed in the basic Federal-aid highway legislation, each State prepares its own specifications for Federal-aid highway construction, subject to approval by Public Roads. However, these specifications will undoubtedly be of interest to specifications writers in particular and to most engineers in highway design and construction, as well as to engineering students.

Highway Statistics, 1959

The Bureau of Public Roads has recently published *Highway Statistics, 1959,* the fifteenth in this annual series. The new bulletin presents the 1959 statistical and analytical tables of general interest on motor fuel, motor vehicles, highway-user taxation, State and local highway finance, highway mileage, and Federal aid for highways. The 153-page bulletin may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D.C., at $1.00 per copy. Earlier annual issues of the series, and a summary of Highway Statistics to 1955, are available from the Superintendent of Documents as indicated on the inside back cover of PUBLIC ROADS.

Manual on Uniform Traffic Control Devices

The Bureau of Public Roads is receiving many requests regarding the availability of the new edition of the *Manual on Uniform Traffic Control Devices for Streets and Highways,* the preparation of which was undertaken last year. It is expected that the printing will be completed about June 30.

U.S. GOVERNMENT PRINTING OFFICE: 1961

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Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1958: a reference guide outline. 75 cents.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 6, 1961.

Contents of this publication may be reprinted. Mention of source is requested.

Final Report of Tests of Concrete Containing Portland Blast-Furnace Slag Cement

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported[1] by William E. Grieb
and George Werner,
Highway Research Engineers

Because portland blast-furnace slag cement is being used increasingly as an alternate for ordinary portland cement, the Bureau of Public Roads several years ago undertook a laboratory study of the strength, durability and other properties of air-entrained concretes prepared with portland blast-furnace slag cement as compared with concrete made with ordinary portland cement. The final results of that study are reported in this article.

The comparisons showed that slag cement concrete had lower early strength but higher ultimate strength than comparable portland cement concrete, according to both compressive and flexural strength tests. Slag cement concrete also had greater durability, under laboratory freezing and thawing tests. There was little difference between the two types in resistance to scaling, in outdoor tests where calcium chloride was used for de-icing.

Tests under intermittent curing conditions indicated that the slag cement concrete may require more care and longer initial moist curing; but given adequate curing, the slag cement concrete tolerated greater variations in the amount of mixing water used. Slag cement with sufficiently high slag content may be suitable for use with highly alkali-reactive aggregate, since the slag content is beneficial in controlling the alkali-aggregate reaction.

Introduction

A PRELIMINARY REPORT on tests of concrete containing portland blast-furnace lag cement, conducted by the Bureau of Pub-ic Roads, was published in 1957.[2] Tests were nade of the physical properties of air-entrained oncretes prepared with five type I portland ements and five type IS portland blast-urnace slag cements, obtained from five lifferent cement plants. At each plant, the ame clinker was used in the manufacture of oth the portland cement and the portland last-furnace slag cement furnished for the ests. By way of definition, portland blast-urnace slag cement is an intimately interground mixture of portland cement clinker nd granulated blast-furnace slag.

The preliminary report presented the results f all tests that had been completed at that ime, including results of tests of compressive nd flexural strengths and sonic moduli of lasticity at ages ranging from 3 through 90 lays, on moist-cured concrete prepared with 5½, 6½, and 7½ gallons of water per bag of ement. Results of strength tests after 28 lays of intermittent curing, and preliminary lata on drying shrinkage and laboratory freez-ng and thawing tests were also reported.

All tests which were still in progress at the time of the preliminary report have now been completed. Strength tests have been made through ages of 1 year for moist-cured specimens and through 90 days for intermittently cured specimens. Freezing and thawing tests were continued through 300 cycles. Volume change measurements were made through a period of 1 year. Slabs placed in outdoor exposure, and subjected to natural freezing and then thawed with commercial flake calcium chloride, were rated for resistance to scaling at the end of two winters. Tests on a 4½-gallon mix were begun and completed after the publication of the preliminary report.

This final report contains all data published in the preliminary report, plus all data that have been obtained since that time. To avoid frequent repetition of the full names of the two types of cement used in the tests, portland cement, type I, will hereafter be referred to simply as portland cement (or type I); and portland blast-furnace slag cement, type IS, will be referred to as slag cement (or type IS).

Conclusions

The conclusions obtained from the results of the continued series of tests support those given in the preliminary report. All of the conclusions derived from the study are combined in this final report, and are presented in the following paragraphs.

Concrete specimens prepared with the slag cements, and continuously moist cured, had lower early strengths than those prepared with portland cements. However, at ages of 90 days and 1 year, the slag cement concretes had higher strengths, and although no tests at ages greater than 1 year were made, the ultimate strength of the slag cement concretes probably will continue to exceed those of the concretes prepared with portland cement. The same trends were shown by both the compressive and flexure strength tests.

Concretes prepared with both types of cement and subjected to intermittent curing, which included 1 or 7 days of initial moist curing, had less strength, in most cases, than duplicate concretes which were continuously moist cured until tested. The concretes prepared with slag cement were affected more by this partial curing than those prepared with portland cement. This is taken to indicate that slag cement concrete may require more care and longer initial moist curing than portland cement concrete.

At an age of 1 year, although the strengths of both types of concretes decreased with increasing amounts of water per bag of cement, the strengths of the concrete prepared with slag cement appeared to be affected less by changes in the water-cement ratio than the strengths of concretes prepared with portland cement. This indicates that concrete prepared with slag cement, provided it is given adequate curing, will tolerate greater variations in the amount of mixing water used and still give more uniform strength than will portland cement concrete.

Concretes prepared with slag or portland cements from the same plant had practically the same amount of shrinkage on drying.

The sonic and static moduli of elasticity of slag cement concrete were slightly lower at early ages and higher at later ages than those for portland cement concrete.

The amount of air-entraining agent needed to entrain 5½ percent of air in concrete was not uniform for the slag cements from the different sources. This varied from the same amount as was used with the corresponding portland cements to about 2½ times that quantity.

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[1] Presented at the 40th annual meeting of the Highway esearch Board, Washington, D.C., January 1961.

[2] PUBLIC ROADS, vol. 29, No. 10, October 1957, pp. 227-232.

Table 1.—Chemical composition [1] and physical properties of cements

	Source A cement		Source B cement		Source C cement		Source D cement		Source E cement	
	Type I	Type IS	Type I	Type IS	Type I	Type IS	Type I	Type IS	Type I	Type IS
Chemical composition (percent):										
Silicon dioxide	20.2	25.0	20.8	26.2	21.1	25.5	21.0	26.7	21.3	26.1
Aluminum oxide [2]	6.0	8.7	5.1	7.8	5.1	6.8	5.6	7.6	5.6	6.7
Ferric oxide	2.8	2.1	3.1	2.3	2.1	1.7	3.3	2.9	3.5	2.4
Calcium oxide	63.4	53.0	63.6	53.6	65.5	59.2	63.4	54.6	62.5	55.7
Magnesium oxide	2.7	3.7	2.8	3.6	2.4	1.8	2.2	3.3	1.7	2.8
Sulfur trioxide	2.3	2.0	1.8	2.3	1.3	1.4	1.9	2.3	2.3	2.2
Loss on ignition [3]	1.5	3.0	1.6	2.0	1.4	2.2	1.2	1.0	1.3	2.2
Sodium oxide	.08	.12	.09	.09	.04	.09	.10	.10	.15	.15
Potassium oxide	.36	.38	.15	.20	.43	.82	.17	.18	1.02	.91
Equivalent alkalies as Na_2O	.32	.37	.19	.22	.32	.61	.21	.22	.82	.75
Phosphorus pentoxide [4]	.10	.08	.03	.02	.03	.04	.09	.05	.04	.03
Manganic oxide	.53	.87	.51	.85	.03	.08	.29	.32	.07	.50
Sulfide sulfur	.09	.56	.10	.68	.01	.33	.08	1.04	.03	.40
Insoluble residue	.28	.57	.16	.73	.22	.68	.08	.20	.20	.26
Titanium dioxide	.27	.44	.24	.28	.28	.28	.18	.20	.24	.44
Free lime	.77	.26	1.20	.46	1.60	.57	.39	.19	.56	.24
Chloroform-soluble organic substances	.003	.005	.003	.006	.003	.002	.003	.005	.003	.008
Calculated compounds (percent):										
Tricalcium silicate	54	--------	57	--------	65	--------	51	------	43	--------
Dicalcium silicate	17	--------	17	--------	11	--------	22	--------	28	--------
Tricalcium aluminate	11	--------	8	--------	10	--------	9	--------	9	--------
Tetracalcium aluminoferrite	8	--------	9	--------	6	--------	10	--------	11	--------
Calcium sulfate	3.9	--------	3.1	--------	2.2	--------	3.2	--------	3.9	--------
Merriman sugar test:										
Neutral point ml.	31.4	3.4	17.7	6.9	46.2	19 2	5.5	2.2	8.3	4.2
Clear point ml.	44.9	4.0	23.3	8.0	68.0	26.2	6.2	2.2	9.7	5.3
Calcium sulfate [5] in hardened mortar as SO_3 g./l.	------	.00	------	.00	.00	.00	.00	--------	.00	.00
Physical properties:										
Apparent specific gravity	3.14	2.99	3.12	3.03	3.12	3.05	3.14	3.03	3.14	3.03
Specific surface (Blaine) cm.²/g.	3810	5000	3325	4820	3425	3775	3680	4040	3555	3605
Passing No. 325 sieve percent.	92.4	96.0	87.5	97.6	80.0	82.9	94.4	98.2	96.2	94.9
Autoclave expansion do.	.10	.01	.06	.00	.09	.04	.07	−.02	.04	.00
Normal consistency do.	25.2	27.6	23.8	27.2	20.4	22.6	26.6	31.2	26.5	26.6
Time of setting (Gilmore): Initial hours.	2.9	2.8	3.7	3.4	1.7	2.6	4 2	3.5	2.9	2.8
Final do.	5.1	6.8	5.6	6.2	3.4	3.9	6.5	7.9	4.8	6.2
Compressive strength (1:2.75 mortar):										
At 3 days p.s.i.	2700	2555	1620	1730	1960	1740	2635	1740	2465	1525
At 7 days p.s.i.	3860	4170	2625	2630	3440	2940	3940	2750	3525	2360
At 28 days p.s.i.	5645	6120	4150	5190	5145	5120	5615	4615	4965	3900
Tensile strength (1:3 mortar):										
At 3 days p.s.i.	295	290	280	265	265	240	305	265	320	245
At 7 days p.s.i.	365	420	355	335	360	345	390	340	375	315
At 28 days p.s.i.	450	500	450	445	415	425	420	460	435	455
Mortar air content percent.	11.6	8 4	10.4	7.4	8.4	8.1	10.7	8.3	9.4	9.2
False set determinations (ASTM Method C359–55T):										
Initial penetration mm.	50+	50+	50+	50+	50+	50+	50+	5	50+	30
5-minute penetration mm.	5	4	50+	7	50+	50+	2	0	32	15
8-minute penetration mm.	2	1	50+	4	50+	8	2	0	18	8
11-minute penetration mm.	2	1	49	4	50+	3	1	0	14	6
Remix penetration mm.	50+	50+	50+	50+	50+	50+	50+	14	50+	44
False set determinations (Federal Standard Specification No. 158, Method 2501):										
Initial penetration mm.	36	40+	37	40+	40+	37	40	--------	38	36
5-minute penetration mm.	18	2	30	18	7	8	31	--------	35	34
Drying shrinkage of mortar: [6]										
Shrinkage 28 days percent.	.08	.10	.08	.08	.09	.10	.07	.08	.09	.10

[1] All determinations made in accordance with current ASTM methods for portland and portland blast-furnace slag cement.
[2] Values for aluminum oxide corrected for titanium and phosphorous oxides that are present.
[3] Values for type IS cements determined by ASTM Method C114–58 T, sec. 30.
[4] Determined by spectrophotometer method.
[5] Determined by ASTM Method C 265–55 T.
[6] Determined by ASTM Specification C 340–55 T.

Concretes prepared with the slag cements for both the 5½- and 6½-gallon mixes had greater durability, as measured by laboratory freezing and thawing tests, than comparable portland cement concretes.

There was no appreciable difference in the resistance to scaling between the concretes prepared with slag cements and the corresponding portland cements, in outdoor tests where calcium chloride was used for de-icing.

Mortars prepared with slag cements usually produced less expansion when used with reactive aggregates than similar mortars prepared with the corresponding portland cements. The amount of slag used in the manufacture of the type IS cement is beneficial in controlling alkali-aggregate reaction. The best results are obtained with a low alkali cement containing a high percentage of slag. Slag cement with a sufficiently high slag content may be suitable for use with highly alkali-reactive aggregate.

Materials and Proportions

The chemical analyses and physical properties of all cements used in the tests are given in table 1. Not included in the previous report but now presented in this table are values for phosphorous pentoxide, titanium dioxide, free lime, and calcium sulfate in hardened mortar. The values for aluminum oxide were corrected for phosphorous pentoxide and titanium dioxide and there are some slight changes, from those previously published, in the loss on ignition and in some of the calculated compound values. Table 1 also includes the results of tests for the false setting properties of the cements, using ASTM Method C 359–55 T and Federal Standard Specification No. 158, Method 2501; and drying shrinkage tests on mortars, using ASTM Spec. C 340–55 T.

The grading and physical properties of the fine and coarse aggregates used are given in table 2, and are the same as in the previous report. The mix data given in table 3 cor
spond with those given in the previous rep
except that data for the 4½-gallon mix a
included.

The mixes were designed on a water-ceme
ratio basis for air-entrained concrete havi
5½-percent air, 2- to 3-inch slump, and
b/b$_0$ (workability factor) of 0.72. The ceme
contents were approximately 7.5, 6.1, 4.9, a
4.1 bags per cubic yard of concrete for wat
contents of 4½, 5½, 6½, and 7½ gallons p
bag of cement. With one exception, the sa
mix proportions were used for all mixes havi
the same water content. This exception w
the mix containing 6½ gallons of water wi
slag cement from source D. A slight chan
was made in that mix in order to maintain t
desired consistency.

Air was entrained in the concrete by the u
of a commercial solution of neutralized Vins
resin. The average amount of air-entraini
solution needed for 5½-percent air for t

able 2.—Grading and physical properties of the aggregates

Grading and physical properties	White Marsh sand	Riverton limestone
Grading: Percentage passing sieves:		
1-inch	------	100
¾-inch	------	70
½-inch	------	40
⅜-inch	100	24
No. 4	96	0
No. 8	79	------
No. 16	66	------
No. 30	50	------
No. 50	23	------
No. 100	7	------
Fineness modulus	2.79	7.06
Physical properties:		
Bulk specific gravity:		
Dry	2.63	2.78
Saturated surface dry	2.65	2.79
Absorption ... percent	0.4	0.4
Strength ratio (Ottawa sand):		
Compressive strength:		
At 7 days ... percent	158	------
At 28 days ... do	167	------
Tensile strength:		
At 7 days ... do	106	------
At 28 days ... do	116	------
Los Angeles wear test, grading A, loss ... percent	------	20.2
Accelerated soundness, Na_2SO_4, loss ... percent	------	3.5

ncretes containing the slag cements was the me as was used with the portland cements om sources C and E, 1½ times as much for ose from sources A and D, and 2½ times as uch as that from source B.

Mixing, Molding, and Curing of Specimens

The mixing, molding, curing, and testing of ecimens was done in accordance with the plicable AASHO or ASTM methods. All ixing was done in an open pan-type labora- ry mixer of 1¾-cubic foot capacity. Both ne and coarse aggregates were used in a wet ndition and the amount of mixing water eded for each batch of concrete was cor- cted for the free water in the aggregates. he following mixing procedure was em- oyed: The cement and the wet sand were ixed for 30 seconds and part of the required nount of water and all of the air-entraining lution were then added and mixed for 30 conds. The wet coarse aggregate was then added, with the necessary amount of additional water. The concrete was mixed for 2½ minutes after the addition of the coarse aggregate. Determinations were made of the slump and unit weight, and of air content by the pressure method. The portions of the concrete used for the slump and unit weight tests were returned to the concrete in the mixer, and were remixed for 15 seconds before the test specimens were prepared.

All beams were made in individual molds. Beams and cylinders that were to be tested after continuous moist curing were molded, cured, and tested in accordance with applicable AASHO procedures.

Beams for freezing and thawing in water by the slow cycle procedure described in ASTM Method C 292 were moist cured for 7 days, cured for 14 days in laboratory air at 73° F. and 50-percent relative humidity, and immersed for 7 days in water at 73° F.

Cylinders and beams that were cured intermittently for strength tests, and beams that were cured intermittently for shrinkage tests, were stored in moist air or water at 73° F. and in laboratory air at 73° F. and 50-percent relative humidity.

Specimen slabs 16 by 24 by 4 inches were made for outdoor exposure to determine the resistance of the concrete to scaling caused by freezing and the removal of ice by calcium chloride. A raised edge was cast around the perimeter of each slab. After being moist cured for 28 days under standard conditions, the slabs were placed in the exposure plot for freezing.

During mixing, premature stiffening was noted in several of the mixes containing slag cement. Tests had been made in accordance with ASTM Method C 359–55 T and Federal Standard Specification No. 158, Method 2501, to measure this false setting of the cement, and their results are given at the bottom of table 1. The false set is indicated in the ASTM procedure by loss of nearly all of the penetration during the 11-minute test period, while in the Federal method a difference between initial and final penetration of more than 17 millimeters after 5 minutes is considered to be an indication of false set. The two methods can be considered as indicating that both types of cement from source A are false setting cements, and that those from source E do not have false set. Both cements from source C would be considered as equally false setting by the Federal method, but the ASTM method shows only the slag cement from that source as having false set. The slag cement from source D could not be tested properly by either method because a plastic mix could not be prepared with the maximum amount of water permitted. While the results of these tests are not conclusive, in general there is a greater tendency toward false setting shown for the slag cements than for the corresponding portland cements.

Discussion of Test Results

The results of strength tests for the continuously moist-cured specimens and the intermittently cured specimens are shown in tables 4–7 and in figures 1–3. Data for the sonic and static moduli of elasticity are shown in tables 8 and 9. The average values for drying shrinkage tests are shown in table 10 and in figure 4, and the laboratory freezing and thawing data are shown in table 11 and in figure 5. These tables and figures are similar to those included in the preliminary report except that these present complete data. Additional data are given in two new tables: Table 12 contains the results of tests on concrete slabs subjected to outdoor freezing and subsequent thawing by the use of calcium chloride; and table 13 gives the results of the mortar bar expansion tests for alkali-aggregate reaction.

These data are discussed in the following text by comparing the results of tests of concrete containing slag cement with those of concrete containing portland cement. In the discussion, emphasis is given to the results not covered in the preliminary report.

Compressive Strength

Table 4 shows the results of the compressive strength tests of the concretes (moist cured until tested) containing the portland cements and the slag cements from the five sources. Each value in the table is the average of five tests made on 6- by 12-inch cylinders. Mixes

Table 3.—Mix data for laboratory specimens [1]

Cement		4½ gal. of water per bag of cement				5½ gal. of water per bag of cement				6½ gal. of water per bag of cement				7½ gal. of water per bag of cement			
Source	Type	Cement content	Slump	Air	Vinsol resin added	Cement content	Slump	Air	Vinsol resin added	Cement content	Slump	Air	Vinsol resin added	Cement content	Slump	Air	Vinsol resin added
		Bags/ cu. yd.	*Inches*	*Percent*	*Ml./bag*	*Bags/ cu. yd.*	*Inches*	*Percent*	*Ml./bag*	*Bags/ cu. yd.*	*Inches*	*Percent*	*Ml./bag*	*Bags/ cu. yd.*	*Inches*	*Percent*	*Ml./bag*
A	I	7.53	3.0	5.3	30.3	6.07	2.9	5.5	18.8	4.94	2.6	5.4	20.6	4.15	2.1	5.7	20.8
A	IS	7.49	2.9	5.4	44.4	6.06	2.8	5.3	25.2	4.93	2.7	5.6	32.3	4.12	2.3	5.6	32.3
B	I	7.52	3.1	5.4	27.0	6.06	2.9	5.6	15.8	4.93	2.4	5.6	19.7	4.15	2.2	5.4	19.4
B	IS	7.50	2.9	5.3	65.1	6.04	2.7	5.2	38.2	4.91	2.6	5.5	48.3	4.14	2.3	5.2	49.3
C	I	7.50	3.5	5.6	25.5	6.09	3.3	5.4	18.1	4.94	2.4	5.5	20.7	4.16	2.1	5.4	23.2
C	IS	7.50	3.4	5.5	27.0	6.07	3.2	5.5	17.8	4.93	2.2	5.5	22.0	4.14	2.3	5.7	23.2
D	I	7.51	2.7	5.7	29.6	6.07	2.3	5.6	17.7	4.95	2.1	5.4	21.0	4.14	1.9	5.6	21.4
D	IS	7.51	1.9	5.5	56.8	6.06	1.3	5.5	26.5	[2] 5.14	2.0	5.4	29.2	4.15	1.6	5.7	30.7
E	I	7.52	2.9	5.4	28.1	6.09	2.8	5.4	19.3	4.94	2.2	5.5	22.2	4.15	2.0	5.4	23.9
E	IS	7.49	2.6	5.6	37.8	6.06	2.2	5.5	21.6	4.94	2.0	5.3	22.8	4.15	1.8	5.5	24.8

[1] Each value is an average of 5 tests. Proportions by oven-dry weight (except as indicated in footnote 2): 4½-gallon mix=94–135–255, 5½-gallon mix=94–190–315, 6½-gallon mix=94–255–), and 7½-gallon mix=94–325–460.

[2] Proportions by oven-dry weight: 94–240–375.

Table 4.—Compressive strength tests on moist-cured specimens [1]

Cement		4½ gallons of water per bag of cement: Compressive strength after—					5½ gallons of water per bag of cement: Compressive strength after—				
Source	Type	3 days	7 days	28 days	90 days	1 year	3 days	7 days	28 days	90 days	1 year
		p.s.i.	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*
A	I	3,440	5,050	6,730	7,590	8,700	2,610	4,030	5,620	6,440	6,840
A	IS	3,310(96)	4,890(97)	7,370(110)	8,360(110)	9,360(108)	2,340(90)	3,810(95)	5,940(106)	7,140(111)	7,840(115)
B	I	2,670	4,570	6,720	7,830	8,400	1,940	3,330	5,400	6,380	6,730
B	IS	2,800(105)	4,280(94)	7,250(108)	8,870(113)	9,850(117)	1,940(100)	3,020(91)	5,790(107)	7,640(120)	8,490(126)
C	I	3,010	4,770	6,060	7,040	8,080	2,070	3,710	5,070	5,500	6,130
C	IS	2,430(81)	4,240(89)	6,270(103)	7,510(107)	8,430(104)	1,720(83)	3,220(87)	5,320(105)	6,390(116)	7,070(115)
D	I	3,610	5,270	6,940	7,610	8,750	2,650	4,160	6,050	6,920	7,310
D	IS	2,750(76)	4,200(80)	7,210(104)	8,340(110)	9,680(111)	2,110(80)	3,390(81)	6,150(102)	7,600(110)	8,330(114)
E	I	3,530	5,000	6,200	6,770	7,520	2,680	4,150	5,660	6,240	6,820
E	IS	2,480(70)	3,440(69)	5,580(90)	6,290(93)	7,490(100)	1,830(68)	2,790(67)	4,460(79)	5,900(95)	6,740(99)
Average, type I		3,250	4,930	6,530	7,370	8,290	2,390	3,880	5,560	6,300	6,700
Average, type IS		2,750(85)	4,210(85)	6,740(103)	7,870(107)	8,960(108)	1,990(83)	3,250(84)	5,530(99)	6,930(110)	7,690(114)

Cement		6½ gallons of water per bag of cement: Compressive strength after—					7½ gallons of water per bag of cement: Compressive strength after—				
Source	Type	3 days	7 days	28 days	90 days	1 year	3 days	7 days	28 days	90 days	1 year
		p.s.i.	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*
A	I	2,050	3,260	4,670	5,440	5,590	1,500	2,350	3,710	4,130	4,200
A	IS	1,770(86)	3,060(94)	4,960(106)	6,170(113)	6,910(124)	1,320(87)	2,250(96)	3,780(102)	4,720(114)	5,310(126)
B	I	1,510	2,580	4,180	5,190	5,640	1,000	1,740	2,980	3,620	3,800
B	IS	1,440(95)	2,310(90)	4,560(109)	6,460(124)	7,160(127)	1,070(107)	1,770(102)	3,700(124)	4,970(137)	5,770(152)
C	I	1,770	2,950	4,100	4,650	4,920	1,320	2,240	3,250	3,640	3,820
C	IS	1,360(77)	2,430(82)	4,240(102)	5,030(108)	5,870(119)	1,010(77)	1,810(81)	3,160(97)	3,870(106)	4,350(114)
D	I	2,180	3,400	4,970	5,780	6,110	1,600	2,350	3,810	4,580	4,640
D	IS	1,630(75)	2,440(72)	4,860(98)	6,110(106)	7,060(116)	1,230(77)	2,120(90)	3,670(96)	4,720(103)	5,360(116)
E	I	2,200	3,310	4,790	5,320	5,460	1,750	2,760	3,860	4,150	4,190
E	IS	1,350(61)	2,220(67)	3,680(77)	4,770(90)	5,820(106)	1,010(58)	1,620(59)	2,780(72)	3,600(87)	4,390(105)
Average, type I		1,940	3,100	4,540	5,280	5,540	1,430	2,290	3,520	4,020	4,130
Average, type IS		1,510(78)	2,490(80)	4,460(98)	5,710(108)	6,560(118)	1,130(79)	1,910(83)	3,420(97)	4,380(109)	5,040(122)

[1] Figures in parentheses represent ratios (in percent) of the strength of the concrete containing type IS cement to the strength of the corresponding concrete containing type I cem t. Each value is an average of 5 tests. Specimens, capped with neat Lumnite cement, were 6- by 12-inch cylinders stored in moist air until tested.

containing 4½, 5½, 6½, and 7½ gallons of water per bag of cement were tested at ages of 3, 7, 28, 90, and 365 days. The ratios (expressed as percentages) of the strengths of the concretes prepared with the slag cements to the strengths of the concretes prepared with the corresponding portland cements are also given. Comparisons of the average compressive strengths of the concretes prepared with the slag cements from all sources with the concretes prepared with the portland cements for each mix are shown (in solid lines) in figure 1.

The data in table 4 show that the concretes prepared with the slag cements from all five sources had lower strengths in most cases at 3 and 7 days than the concretes prepared with the corresponding portland cements. The only exceptions were some of the mixes prepared with the cements from source B. At 28, 90, and 365 days, the concretes prepared with the slag cements from sources A, B, C, and D had greater compressive strengths, in all except 3 of the 48 cases, than concretes prepared with the portland cements from the same four sources. Concrete prepared with slag cement from source E had lower compressive strengths at all ages up to and including 90 days than the corresponding concrete prepared with the portland cement, but at 365 days equal or greater strengths were obtained. As shown in figure 1, the average compressive strengths for concrete prepared with slag cement from all five sources were lower at 3 and 7 days, approximately the same at 28 days, and higher at 90 and 365 days, than the compressive strengths of concrete prepared with portland cement.

Flexural Strength

The results of the flexural strength tests of the concretes (moist cured until tested), prepared with the portland cements and the slag cements from the five sources, are given in table 5. Each value is the average of five tests made on 6- by 6- by 21-inch beams. Mixes containing 4½, 5½, 6½, and 7½ gallons of water per bag of cement were tested at 3, 7, 28, 90, and 365 days. The ratios (expressed as percentages) of the flexural strengths of the concretes prepared with the slag cements to the flexural strengths of the concretes prepared with the corresponding portland cements are also given. Comparisons of the average flexural strengths of the concretes (from all sources) prepared with the slag cements with concretes prepared with the portland cements are shown for each mix (in dashed lines) in figure 1.

Approximately the same trends developed for flexural strength as for compressive strength. In most cases the concrete prepared with the slag cements had lower flexural strengths at 3 and 7 days than the corresponding concretes prepared with the portland cements. The concretes prepared with the slag cements from sources A, B, C, and D had greater flexural strengths at 28, 90, and 365 days than the corresponding portland cem t concrete in all cases but one. Concretes p- pared with the slag cement from source E l d lower strengths at 28 days, but equal or grea r strengths at 90 and 365 days, than co- sponding portland cement concretes.

The average flexural strength of the c- cretes prepared with the slag cements from e five sources, as shown in figure 1, was lo r at 3 and 7 days but higher at 28, 90, and 5 days than that for the concretes prepared w h the portland cements.

Effect of Water Content

The relations between the strengths of c- cretes prepared with the two types of ceme s were not much affected by changes in wa r content. At 3, 7, 28, and 90 days there s little difference between the strength rat s for the 4½- and the 7½-gallon mixes. At , 7, and 28 days there was a slight decrease i strength ratios with increases in the wat , while at 90 days there was a slight incre in the strength ratios. At 1 year, the p- centage increase in strength of the slag cem t concrete over the portland cement concr became greater with each increase in wa content. In figure 2, the ratios of the aver strength of the slag cement concretes at ea age tested (combined for all five sources) the average strength of the portland ceme concretes at the same age are plotted agai the water content of the concretes. It will bserved that, for both compressive and flex-ral strengths, at ages through 90 days there as little difference in the strength ratios for he different mixes. At 1 year, the strength atios increased as the water content increased. This is interpreted to mean that the ultimate trengths, as represented by the strengths at year, of the concrete prepared with slag ement were less affected by changes in water ontent than the strengths of concrete repared with portland cement.

Effect of Intermittent Curing on Strength

The effect of intermittent or partial curing n the compressive and flexural strengths of oncretes prepared with the portland cements nd the slag cements is shown in tables 6 and , and average values for all cements are hown in figure 3. One mix, containing 5½ allons of water per bag of cement, was used n these tests. Three groups of intermittently ured specimens were tested at 28 days and wo groups at 90 days.

Of those tested at 28 days, the first group as moist cured for 1 day, then stored in boratory air at 73° F. and 50-percent relative umidity for 27 days, and tested dry. The econd group was moist cured for 1 day, llowed by 26 days storage in laboratory air, en immersed in water for 1 day, and tested et. The third group was moist cured for 7 ays, followed by 20 days in laboratory air, en immersed in water 1 day, and tested wet. The fourth group, one of the two tested at) days, was moist cured for 1 day, followed y 88 days in laboratory air, then immersed water 1 day, and tested wet. The fifth oup was moist cured for 7 days, followed by 2 days in laboratory air, then immersed in ater 1 day, and tested wet.

The results of the compressive strength sts on these specimens and on specimens ntinuously moist cured for the same time riod (28 and 90 days) are given in table 6. he ratios (expressed as percentages) of the mpressive strengths of the intermittently red specimens to the strengths of duplicate ntinuously moist-cured specimens are also ven.

Of the 28-day specimens, those in group 1 owed (fig. 3) an average reduction in rength of 30 percent for the portland cement ncrete and 31 percent for the slag cement ncrete when compared with the compressive rengths of similar continuously moist-cured ecimens. Group 2 showed a loss in strength 36 percent for the portland cement concrete d 41 percent for the slag cement concrete. roup 3 showed losses of 10 and 13 percent, spectively. Most of the 28-day specimens repared with portland cements and intermittently cured showed a smaller average duction in compressive strength than milarly cured specimens prepared with slag ments. However, these differences were ry slight. The specimens in group 1 had igher strengths for both the portland and the ag cement concretes than the specimens in oup 2.

Of the 90-day specimens, those in group 4 showed an average reduction in strength of 44 percent for the portland cement concrete and 52 percent for the slag cement concrete. Those in group 5 showed losses of 22 and 28 percent, respectively. All of the 90-day intermittently cured specimens prepared with portland cement showed less reduction in strength than the corresponding specimens prepared with slag cement. The average percentage reduction in compressive strength for the slag cement specimens tested at 90 days was greater than for the similarly cured slag cement specimens tested at 28 days.

For the intermittently cured specimens of the portland cement concretes and the slag cement concretes which were moist cured the same length of time, the actual compressive strengths were approximately the same for those tested at 28 days as for those tested at 90 days. This appears to indicate that the dry curing was not beneficial in development of strength.

The results of tests for flexural strength of intermittently cured specimens are shown in table 7. These data indicate the same general trends as were shown for compressive strength. The portland cement concretes showed less reduction in flexural strength due to intermittent curing than the corresponding slag cement concretes. This difference between the reductions shown for the portland cement and the slag cement concretes was greater for the flexural strength tests than for the compressive strength tests. Concrete prepared with four of the five portland cements and one of the slag cements and tested at 28 days developed as much or more strength when given only 7 days initial moist curing and immersed in water 24 hours prior to testing as the concrete which was moist cured continuously for 28 days. The 90-day tests did not show this feature.

The average results of these tests of intermittent curing are summarized in figure 3. The tests indicate that concrete prepared with slag cement may be more susceptible to defective curing practices than concrete prepared with portland cement. It is, of course,

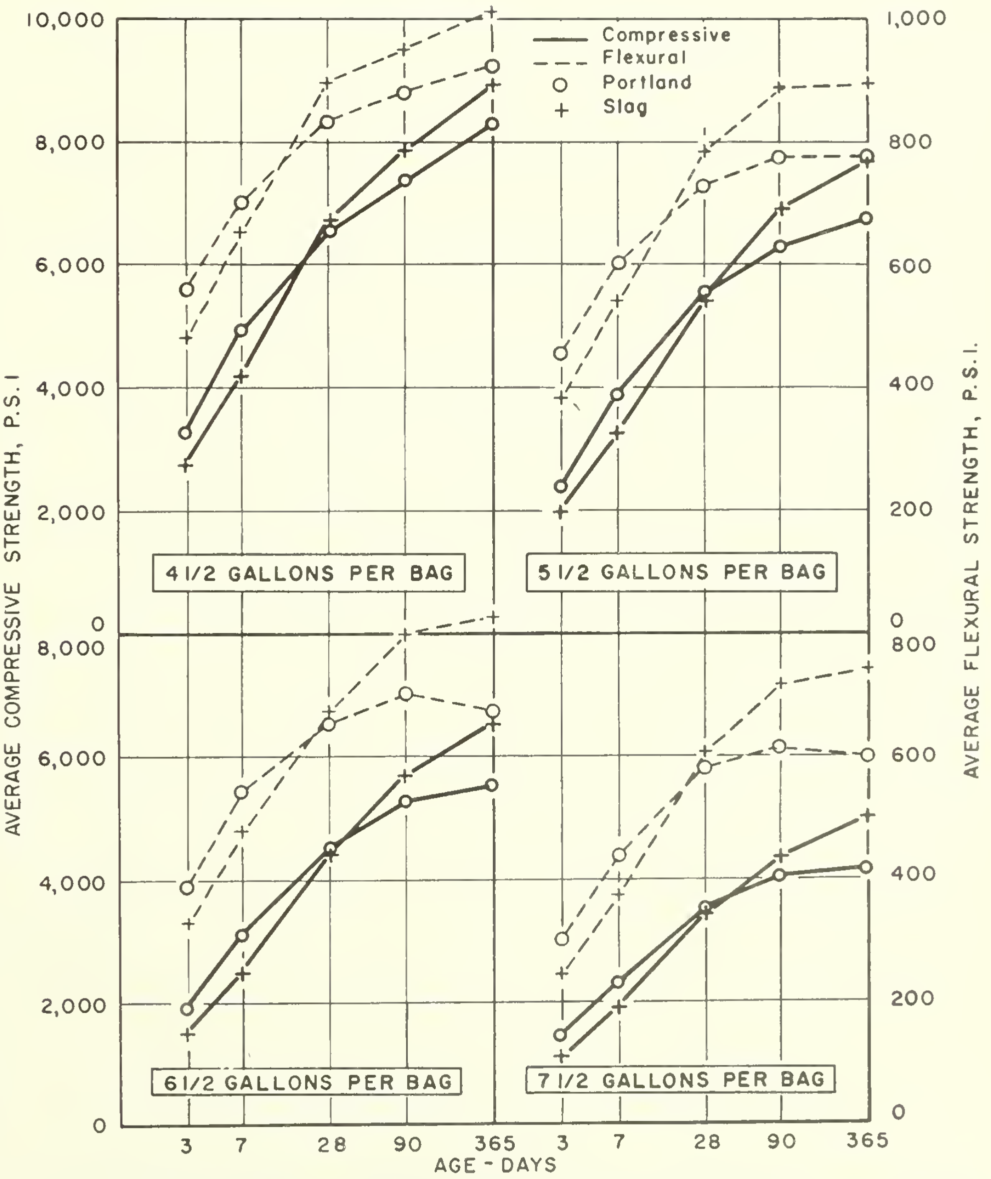

Figure 1.—Compressive and flexural strengths of concretes made with portland and slag cements.

Table 5.—Flexural strength tests on moist-cured specimens [1]

Cement: Source	Cement: Type	4½ gallons of water per bag of cement: Flexural strength after— 3 days	7 days	28 days	90 days	1 year	5½ gallons of water per bag of cement: Flexural strength after— 3 days	7 days	28 days	90 days	1 year
		p.s.i.	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*
A	I	575	700	855	895	955	495	605	730	760	775
A	IS	545(95)	735(105)	925(108)	1,005(112)	1,090(114)	420(85)	610(101)	880(121)	920(121)	925(119)
B	I	505	680	790	915	910	380	570	740	790	810
B	IS	495(98)	635(93)	885(112)	975(106)	1,020(112)	395(104)	520(91)	750(101)	885(112)	935(115)
C	I	500	685	785	780	885	410	550	630	695	690
C	IS	460(92)	705(103)	955(122)	915(117)	950(107)	365(89)	550(100)	730(116)	815(117)	815(118)
D	I	590	705	850	895	955	500	610	750	780	810
D	IS	485(82)	630(89)	875(103)	950(106)	1,020(107)	400(80)	525(86)	775(103)	925(119)	935(115)
E	I	595	740	890	930	905	495	670	805	855	790
E	IS	445(75)	605(82)	875(98)	930(100)	1,010(112)	375(76)	505(75)	790(98)	915(107)	895(113)
Average, type I		555	700	835	885	920	455	600	730	775	775
Average, type IS		485(87)	660(94)	905(108)	955(108)	1,020(111)	390(86)	540(90)	785(108)	890(115)	900(116)

Cement: Source	Cement: Type	6½ gallons of water per bag of cement: Flexural strength after— 3 days	7 days	28 days	90 days	1 year	7½ gallons of water per bag of cement: Flexural strength after— 3 days	7 days	28 days	90 days	1 year
		p.s.i.	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*
A	I	420	560	635	695	655	315	425	590	605	580
A	IS	365(87)	535(96)	715(113)	800(115)	820(125)	270(86)	435(102)	660(112)	730(121)	770(133)
B	I	330	485	695	760	720	240	370	520	645	610
B	IS	305(92)	440(91)	690(99)	800(105)	825(114)	235(98)	365(99)	640(123)	780(121)	815(134)
C	I	345	505	565	600	615	270	430	530	530	540
C	IS	315(91)	475(94)	690(122)	775(129)	755(123)	220(81)	355(83)	555(105)	630(119)	650(120)
D	I	435	555	640	720	690	335	455	600	625	645
D	IS	345(79)	485(87)	730(114)	835(116)	870(126)	275(82)	395(82)	630(105)	760(122)	750(116)
E	I	440	620	730	745	700	355	490	670	635	615
E	IS	310(70)	445(72)	610(84)	780(105)	850(121)	225(63)	340(69)	540(81)	650(102)	745(121)
Average, type I		395	545	655	705	675	305	435	580	610	600
Average, type IS		330(84)	475(87)	685(105)	800(113)	825(122)	245(80)	380(87)	605(104)	710(116)	745(124)

[1] Figures in parentheses represent ratios (in percent) of the strength of concrete containing type IS cement to the strength of the corresponding concrete containing type I cement. Ea value is an average of 5 tests. Specimens, stored in moist air until tested, were 6- by 6- by 21-inch beams tested in accordance with ASTM Method C-78 with third point loading on 18-inch span; side as molded in tension.

desirable to cure all concrete as perfectly as possible. Apparently somewhat more care must be exercised for slag cement concrete to ensure obtaining the desirable features furnished by this type of cement.

Sonic and Static Moduli of Elasticity

The results of tests for sonic moduli of elasticity are given in table 8. Determinations were made on the 6- by 6- by 21-inch beams prior to tests for flexural strength. In general, the same trends as were obtained with the compressive strength tests were shown by the sonic tests. At ages of 3, 7, and 28 days the portland cement concrete had higher average sonic moduli; and at 90 days and 1 year the slag cement concrete had higher average sonic moduli (except for the 4½-gallon mix). The difference in moduli for the two types of cement was not great—usually less than 5 percent.

The data for the static moduli of elasticity tests are shown in table 9. These data are limited to tests at each age for the 4½-gallon mix and to tests at 1 year only for the other mixes. (The static moduli test apparatus was not available for the early age tests on the 5½-, 6½-, and 7½-gallon mixes.) Determinations were made on 6- by 12-inch cylinders by use of an autographic stress-strain recorder with a 6-inch gage length, prior to tests for compressive strength. The same trends were shown for the static moduli as were shown for the sonic moduli. The sonic moduli were about 7 percent higher than the static moduli.

Drying Shrinkage

Tests for shrinkage in drying were made on concrete specimens prepared with each of the portland and slag cements, using mixes with 5½ and 6½ gallons of water per bag of cemen The specimens were 3- by 4- by 16-inch bean with gage studs cast in each end. Thr beams prepared with each cement and wat content were moist cured for 2 days and thr sets were moist cured for 7 days prior to tl beginning of the measurements for shrinkag The specimens were stored in room air

Table 6.—Compressive strength tests on specimens cured intermittently [1]

Cement: Source	Cement: Type	5½ gallons of water per bag of cement: Compressive strength after curing for— 28 days: 1 day moist, 27 days dry [2]	28 days: 1 day moist, 26 days dry, 1 day soak [2]	28 days: 7 days moist, 20 days dry, 1 day soak [2]	28 days: 28 days moist [3]	90 days: 1 day moist, 88 days dry, 1 day soak [2]	90 days: 7 days moist, 82 days dry, 1 day soak [2]	90 days: 90 days moist [3] [4]
		p.s.i.	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*
A	I	4,420 (77)	3,980 (69)	5,100 (88)	5,770	4,000 (62)	5,140 (80)	6,440
A	IS	4,080 (69)	3,760 (63)	5,360 (90)	5,940	3,620 (51)	5,450 (76)	7,140
B	I	3,420 (63)	3,050 (56)	4,660 (86)	5,400	3,190 (50)	4,850 (76)	6,380
B	IS	4,000 (67)	3,270 (55)	4,790 (81)	5,950	3,690 (48)	4,950 (65)	7,640
C	I	3,320 (64)	3,030 (59)	4,720 (91)	5,160	2,970 (54)	4,490 (82)	5,500
C	IS	3,180 (61)	2,840 (55)	4,490 (86)	5,200	2,840 (44)	4,620 (72)	6,390
D	I	4,140 (72)	3,620 (62)	5,260 (91)	5,790	3,810 (55)	5,160 (75)	6,920
D	IS	4,050 (69)	3,450 (59)	5,010 (86)	5,830	3,520 (46)	5,270 (69)	7,600
E	I	4,040 (75)	3,820 (70)	4,960 (92)	5,420	3,850 (62)	4,990 (80)	6,240
E	IS	3,700 (80)	2,870 (62)	4,180 (91)	4,600	2,990 (51)	4,660 (79)	5,900
Average, type I		3,870 (70)	3,500 (64)	4,940 (90)	5,510	3,560 (56)	4,930 (78)	6,300
Average, type IS		3,800 (69)	3,240 (59)	4,770 (87)	5,500	3,330 (48)	4,990 (72)	6,930

[1] Figures in parentheses represent the ratios (in percent) of the strength of the intermittently cured specimens (6- 12-inch cylinders) to the strength of the continuously moist-cured specimens. Each value is an average of 5 tests.

[2] Specimens capped with sulfur cement.

[3] Specimens capped with neat Lumnite cement.

[4] These specimens were made on different days than the 90-day specimens which were cured intermittently. These a the same values as given in table 4.

73° F. and 50-percent relative humidity for 90 days, and length measurements were made frequently. The specimens were then immersed in water at room temperature for 14 days. This procedure was repeated three times.

The average results of the shrinkage tests for concretes prepared with the two types of cement and 5½ and 6½ gallons of water per bag of cement are shown in figure 4, and the shrinkages at the end of the third drying period are given in table 10. These are averages for all concretes prepared with each type of cement. The differences among the concrete specimens prepared with the two types of cement were too small to be significant. It appears that concern over the shrinkage of concrete prepared with slag cement is unfounded. The result of the drying shrinkage tests of mortar bars, ASTM Spec. C 340–55 T, shown at the bottom of table 1, supports the same conclusion.

Laboratory Freezing and Thawing

Freezing and thawing tests were made on both slag and portland cement concretes prepared with 5½ and 6½ gallons of water per bag of cement. Beams measuring 3 by 4 by 16 inches were moist cured for 7 days, followed by 14 days of storage in laboratory air, and then immersed in water for 7 days prior to freezing. Freezing and thawing was in accordance with ASTM Method C 292 for slow freezing and thawing in water. The results of these tests and the durability factors of the concretes are given in table 11 and are shown in figure 5. The durability factor was calculated as follows:

$$DF=(P\times N)\div M$$

Where:

DF = durability factor.

P = relative dynamic modulus of elasticity at N cycles, in percent (60-percent minimum).

N = number of cycles at which P reached 60 percent or 300, whichever is less.

M = 300 (cycles).

These tests show that the concretes prepared with the slag cement from all five sources had better durability, as determined by laboratory freezing and thawing, than the corresponding concretes prepared with the portland cements.

Scaling Tests with Calcium Chloride

In testing the effects of calcium chloride when used for ice removal on portland and slag cement concretes, three mixes were used, containing 5½, 6½, and 7½ gallons of water per bag of cement. Concrete slabs 16 inches wide, 24 inches long, and 4 inches deep were prepared with each type of cement for outdoor exposure tests. They were moist cured for 28 days and then stored in the outdoor exposure area. A dam was cast around the top perimeter of each specimen. The slabs were in the exposure area for about 6 months before freezing weather began.

When freezing was expected, the top surface of each specimen was covered with one-fourth

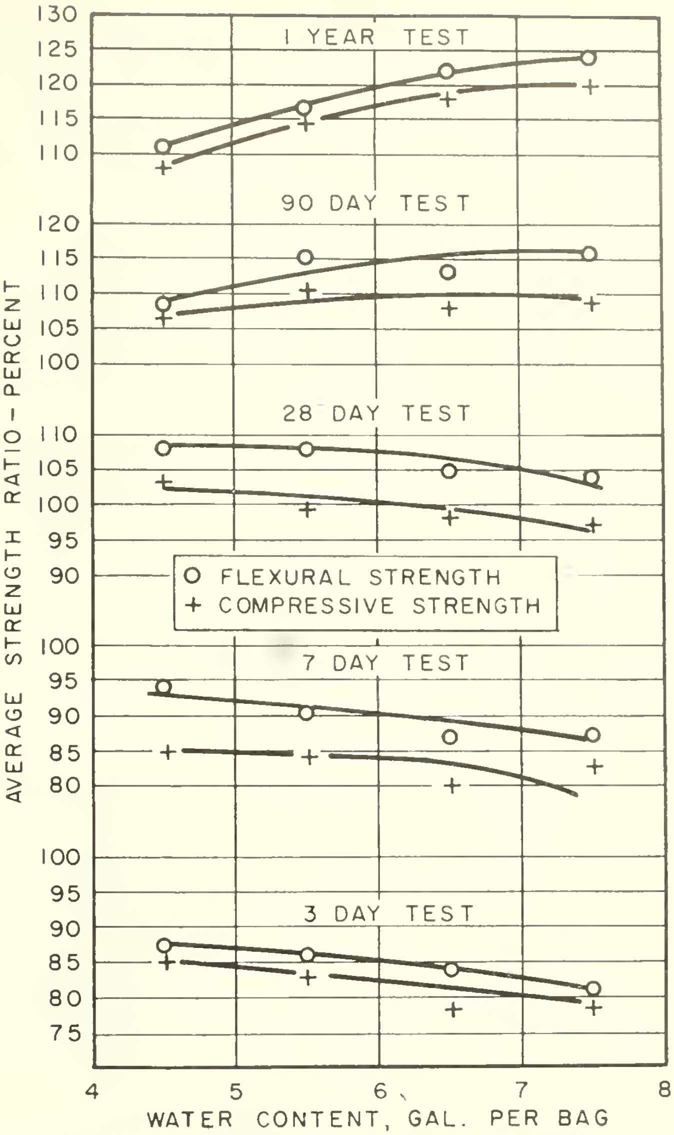

Figure 2.—Effect of water content on ratio of strength of slag cement concrete to strength of portland cement concrete.

Table 7.—Flexural strength tests on specimens cured intermittently [1]

Cement		5½ gallons of water per bag of cement: Flexural strength after curing for—						
		28 days				90 days		
Source	Type	1 day moist, 27 days dry	1 day moist, 26 days dry, 1 day soak	7 days moist, 20 days dry, 1 day soak	28 days moist	1 day moist, 88 days dry, 1 day soak	7 days moist, 82 days dry, 1 day soak	90 days moist [2]
		p.s.i.	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*
A	I	435 (58)	565 (76)	715 (96)	745	600 (79)	710 (93)	760
A	IS	470 (54)	585 (67)	730 (83)	875	535 (58)	720 (78)	920
B	I	445 (63)	425 (60)	750 (106)	710	445 (56)	785 (99)	790
B	IS	470 (63)	515 (69)	785 (105)	745	525 (59)	745 (84)	885
C	I	445 (63)	420 (59)	710 (100)	710	370 (53)	635 (91)	695
C	IS	395 (51)	410 (53)	730 (95)	770	355 (44)	720 (88)	815
D	I	485 (64)	570 (75)	765 (101)	760	575 (74)	780 (100)	780
D	IS	490 (59)	465 (56)	815 (99)	825	485 (52)	830 (90)	925
E	I	480 (63)	570 (75)	785 (103)	765	540 (63)	750 (88)	855
E	IS	435 (55)	465 (59)	720 (91)	790	445 (49)	725 (79)	915
Average, type I		460 (62)	510 (69)	745 (101)	740	505 (65)	730 (94)	775
Average, type IS		450 (56)	490 (61)	755 (94)	800	470 (53)	750 (84)	890

[1] Figures in parentheses represent the ratios (in percent) of the strength of the intermittently cured specimens to the strength of the moist-cured specimens. Each value is an average of 5 tests. Specimens were 6- by 6- by 21-inch beams tested in accordance with ASTM Method C 78 with third point loading on an 18-inch span; side as molded in tension.

[2] These specimens were made on different days than the 90-day specimens which were cured intermittently. These are the same values as given in table 5.

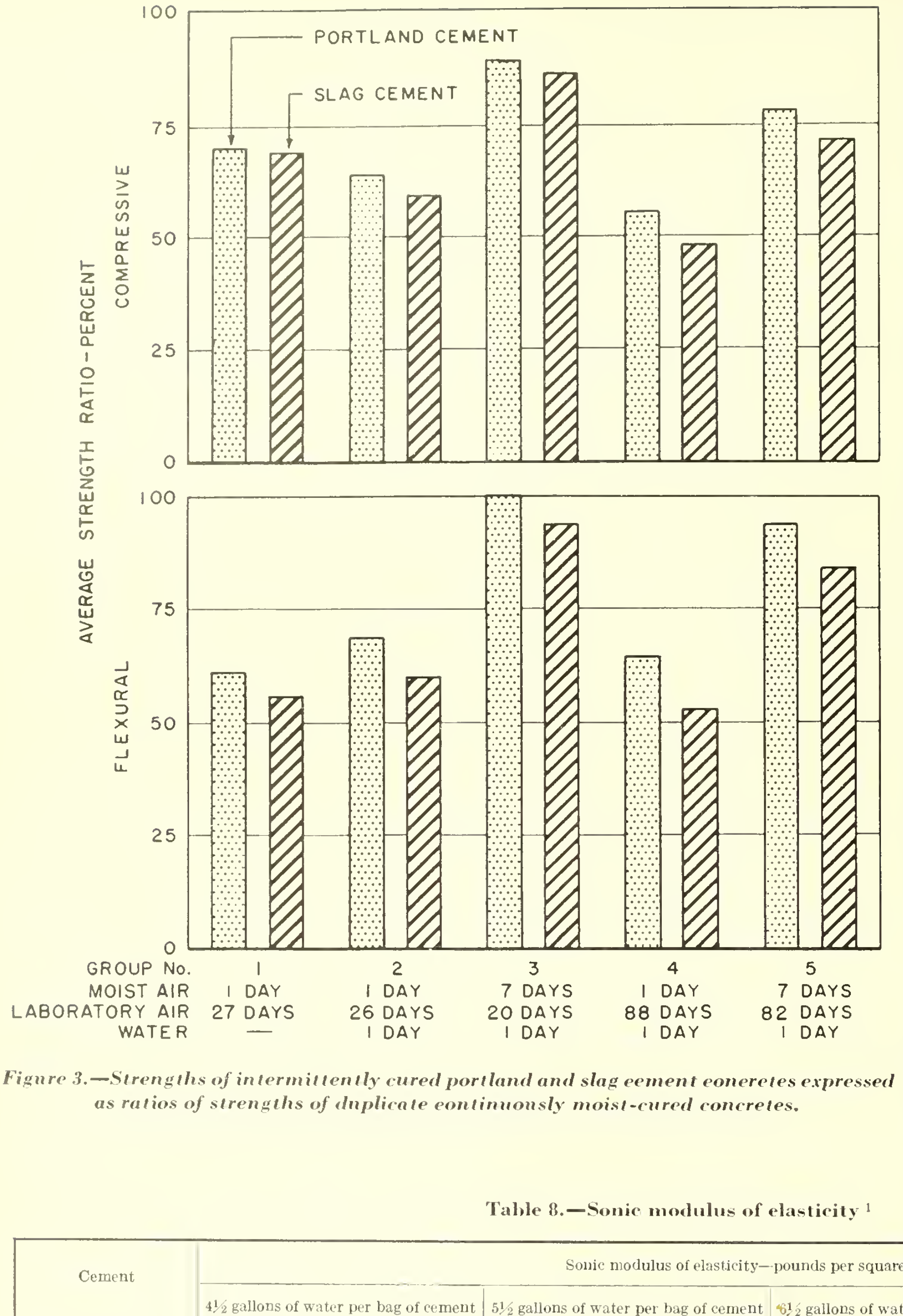

Figure 3.—Strengths of intermittently cured portland and slag cement concretes expressed as ratios of strengths of duplicate continuously moist-cured concretes.

to one-half inch of water. Each morning after the water had frozen, flake calcium chloride was spread over the ice at a rate of 2.4 pounds per square yard of ice-encrusted surface. After the ice had thawed, usually about 3 hours later, the calcium-chloride solution was washed off and the surface was again covered with fresh water. The tests were continued through two winters, with a total of 55 cycles of freezing and thawing.

The slabs were examined periodically and rated by visual observation according to the amount and depth of scaling of the exposed surface. A rating of zero indicated that no scaling had occurred, and a rating of 10 indicated deep scaling over the entire surface of the specimen. A summary of the ratings is shown in table 12.

These tests showed very little difference between the behavior of the portland cement concrete and that of the slag cement concrete. However, considerable differences were found among the cements, both portland and slag, prepared by the different plants, and used in concretes containing 5½ gallons of water per bag of cement. The concrete specimens prepared with cements, both portland and slag, from sources A and B were generally much more resistant to scaling by the de-icing agent than the other concretes. When a water content of 6½ or 7½ gallons per bag was used, severe scaling was found on all specimens. It is apparent that the water content of the concrete is of primary importance in the scaling of concrete caused by de-icing agents. Only when the water content is kept at a low value can differences in the quality of the concrete caused by other factors influence the results obtained.

Mortar Bar Expansion Tests

The alkali-reactivity of both the portland and slag cements was evaluated by the mortar bar tests, as specified in Federal Specification SS-C-208b and ASTM Specification C 340-55 (tests made in accordance with ASTM Method C 227-52 T) for portland-pozzolan cement. These tests involve the determination of the expansion of 1- by 1- by 10-inch

Table 8.—Sonic modulus of elasticity [1]

Cement		Sonic modulus of elasticity—pounds per square inch × 10⁶																							
		4½ gallons of water per bag of cement						5½ gallons of water per bag of cement						6½ gallons of water per bag of cement						7½ gallons of water per bag of cement					
Source	Type	3 days	7 days	28 days	90 days	180 days	1 year	3 days	7 days	28 days	90 days	180 days	1 year	3 days	7 days	28 days	90 days	180 days	1 year	3 days	7 days	28 days	90 days	180 days	1 year
A	I	5.44	6.01	6.64	6.78	7.07	7.25	----	----	6.38	6.76	6.82	6.92	4.95	5.73	6.32	6.66	6.80	6.68	4.18	5.22	5.80	6.07	6.15	6.18
A	IS	5.22	5.38	6.19	6.75	7.01	7.10	----	----	6.17	6.70	6.84	6.91	4.79	5.45	6.04	6.71	6.92	6.87	3.85	4.86	5.72	6.16	6.63	6.66
B	I	4.99	5.70	6.51	6.83	6.95	7.05	----	5.70	6.46	6.52	6.62	6.69	4.44	5.44	6.19	6.57	6.70	6.60	3.66	4.84	5.62	6.00	----	6.13
B	IS	5.03	5.52	6.43	6.95	7.13	7.21	----	4.95	6.11	6.72	6.91	7.09	4.26	4.97	5.86	6.41	6.71	6.78	3.59	4.61	5.51	6.27	----	6.54
C	I	5.28	6.40	6.56	6.57	6.82	6.93	----	5.75	6.33	6.55	6.69	6.78	4.62	5.57	6.15	6.33	6.46	6.52	3.99	5.06	5.83	5.98	----	6.02
C	IS	5.01	6.25	6.39	6.51	6.94	6.99	----	5.65	6.39	6.57	6.67	6.80	4.38	5.42	6.20	6.57	6.76	6.72	3.65	4.72	5.73	6.20	----	6.40
D	I	5.22	6.05	6.60	6.86	7.25	7.01	----	6.08	6.43	6.69	6.75	6.82	4.91	5.69	6.27	6.59	6.72	6.61	4.36	5.19	5.89	6.04	----	6.15
D	IS	4.97	5.68	6.54	6.85	7.00	7.02	----	5.95	6.65	7.01	7.18	7.28	4.66	5.35	6.22	6.63	6.89	6.80	4.00	4.76	5.82	6.33	----	6.42
E	I	5.58	6.12	6.71	6.90	6.94	6.96	----	6.20	6.65	6.78	6.95	7.07	5.06	5.84	6.45	6.69	6.77	6.66	4.56	5.51	6.15	6.19	----	6.23
E	IS	4.95	5.38	6.15	6.50	6.76	6.96	----	5.57	6.27	6.54	6.91	7.19	4.58	5.30	5.97	6.59	6.81	6.84	3.94	4.73	5.67	6.11	----	6.42
Average, type I		5.30	6.06	6.60	6.79	7.01	7.04	----	5.93	6.45	6.66	6.77	6.86	4.80	5.65	6.28	6.57	6.69	6.61	4.15	5.16	5.86	6.06	----	6.14
Average, type IS		5.04	5.64	6.34	6.71	6.97	7.06	----	5.53	6.32	6.71	6.90	7.05	4.53	5.47	6.06	6.58	6.82	6.80	3.81	4.74	5.69	6.21	----	6.49

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[1] Sonic modulus determined on 6- by 6- by 21-inch beams prior to testing for flexural strength. Specimens were continuously moist cured. Each value is an average of 5 tests.

Table 9.—Static modulus of elasticity [1]

Cement		Static modulus of elasticity—pounds per square inch × 10⁶							
		For mixes with 4½ gallons of water per bag of cement					At 1 year,[2] for mixes with water content per bag of cement of—		
Source	Type	3 days	7 days	28 days	90 days	1 year	5½ gal.	6½ gal.	7½ gal.
A	I	4.82	5.89	6.24	6.69	6.66	6.74	6.26	5.70
A	IS	4.63	5.70	6.28	6.86	6.85	6.65	6.68	6.60
B	I	4.53	5.70	6.01	6.65	6.83	6.42	6.48	5.69
B	IS	4.11	5.67	6.63	7.14	6.96	6.62	6.92	6.70
C	I	4.30	5.52	5.92	6.51	6.68	6.42	6.21	5.74
C	IS	4.31	5.61	5.95	6.59	6.76	6.34	6.73	6.19
D	I	4.81	5.91	6.26	6.20	6.65	6.43	6.54	6.26
D	IS	4.38	4.88	7.30	6.93	6.96	7.12	6.70	6.77
E	I	4.90	6.31	6.82	6.39	6.74	6.90	6.69	5.56
E	IS	4.81	5.29	6.06	6.34	6.84	6.60	6.99	6.45
Average, type I		4.67	5.87	6.25	6.49	6.71	6.58	6.44	5.79
Average, type IS		4.45	5.43	6.44	6.77	6.87	6.67	6.80	6.54

[1] Static modulus determined on 6- by 12-inch cylinders prior to testing for compressive strength. Specimens were continuously moist cured. Each value is an average of 5 tests.

[2] No tests were made at 90 days or less since the static modulus test apparatus was not available during that period of the test program.

Table 10.—Drying shrinkage of concrete

Water content	Initial moist curing	Final shrinkage [1]	
		Portland cement concrete	Slag cement concrete
Gal./bag	*Days*	*Percent*	*Percent*
5½	2	0.040	0.041
5½	7	.038	.040
6½	2	.042	.037
6½	7	.040	.039

[1] Shrinkage after 2 cycles of drying for 60 days and 14 days immersion in water, followed by an additional 60 days drying.

mortar specimens prepared with crushed and graded Pyrex glass. The principal difference between the Federal and the ASTM tests is the use of a significantly higher water-cement ratio in the latter. In addition, a modified mortar bar test was made, using the same size of specimen. This was prepared from a 1:2 mortar containing ASTM C 109 Ottawa sand with various amounts of No. 8- to No. 30-size reactive opal, ranging up to 2 percent, and having a water-cement ratio of 0.50. The specimens for each test were stored in moist air at 100° F.

The expansion data for all mortar bar tests are given in table 13. Both Federal Specification SS–C–208b and ASTM Specification C 340–55 limit the expansion of mortar bars to not more than 0.02 percent at 14 days, or 0.06 percent at 8 weeks. Presumably a cement which meets these limits would not be expected to cause excessive expansion in concrete containing alkali-reactive aggregates. The portland cements from sources C and E would be considered as potentially expansive by both the Federal and ASTM procedures, but the portland cement from source A would be similarly classified only by the ASTM procedure. None of the slag cements would be considered as potentially reactive by either procedure, based on the expansions at 8 weeks.

The modified mortar bar test in which opal was used as the reactive material is similar to tests used by numerous investigators to study the various factors which influence the expansion resulting from the alkali-aggregate reaction. Because of differences in reactivity of opal and other naturally occurring materials obtained from different sources, expansion tests using natural aggregates have never been standardized to the point of establishing a definite criterion by which the reactivity of a cement can be judged. Comparison of the expansions shown in table 13 for mortar bars prepared with and without opal indicate that any expansion of more than 0.04 percent can be attributed to a reaction between the alkalies of the cement and the opal.

Perhaps the nearest approach to an applicable criterion for an excessive amount of expansion in this test is found in the specification for concrete aggregates, ASTM C 33–57, in which an expansion of more than 0.10 percent at 6 months is used to define potentially reactive aggregates. Using this criterion, it is found that there is a pessimum amount of opal which will cause an excessive expansion at 6 months with the portland cements from sources A, C, and E and the slag cements from sources C and E. For the cements from sources A, B, D, and E, the expansions of the slag cements were less than those for the portland cements, but the slag cement from source C showed a greater expansion than the portland cement from that source. It should be noted that for this source, the alkali content of the slag cement was nearly double that of the portland cement, thus accounting for the increase in expansion. Apparently, the alkalies present in blast-furnace slag are available for reaction with susceptible aggregates.

The results of the modified mortar bar tests using opal appear to be somewhat in conflict with the results obtained with the ASTM and Federal procedures for determining reactivity. The modified tests indicate that the slag constituent of the high-alkali slag cements is not entirely effective in preventing expansion resulting from the alkali-aggregate reaction under the conditions of this test.

It was noted that two of the slag cements had an equivalent alkali content, as shown in table 1, of over 0.6 percent, which would cause them to be classified as high-alkali cements. It was also noted, in the modified method of

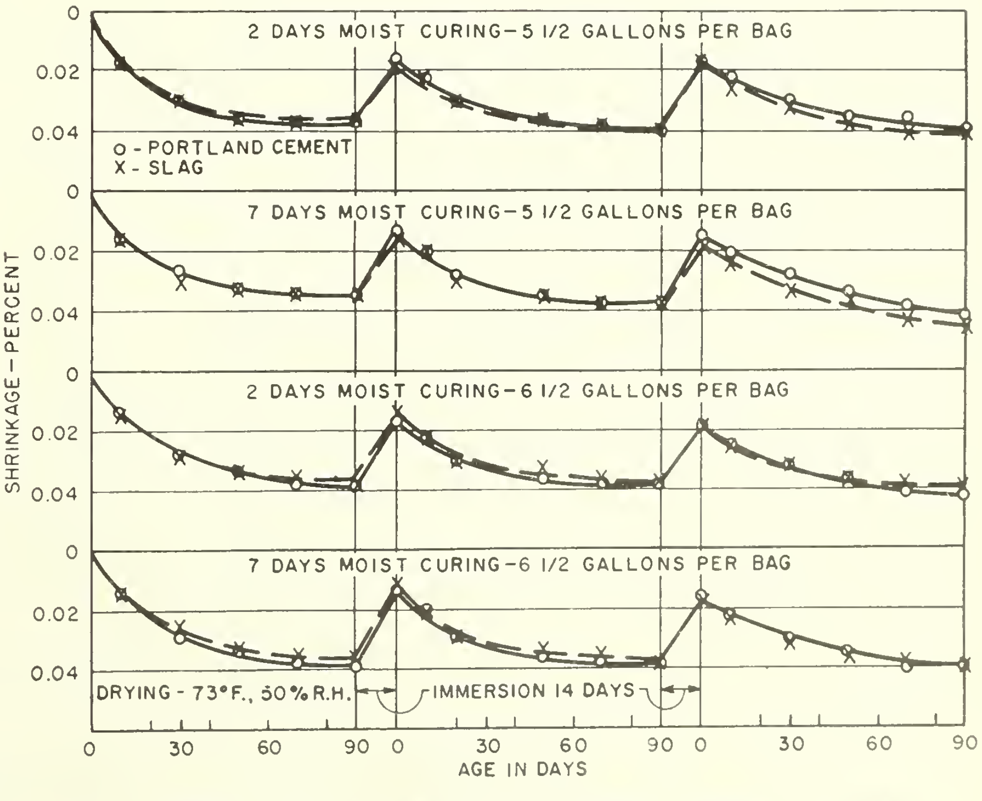

Figure 4.—Rate of drying shrinkage of portland and slag cement concretes.

Table 11.—Results of laboratory freezing and thawing tests[1]

Cement		5½ gallons of water per bag of cement									6½ gallons of water per bag of cement								
Source	Type	Percent of original N^2 after freezing and thawing for indicated cycles								Durability factor	Percent of original N^2 after freezing and thawing for indicated cycles								Durability factor
		10 cycles	20 cycles	50 cycles	100 cycles	150 cycles	200 cycles	250 cycles	300 cycles		10 cycles	20 cycles	50 cycles	100 cycles	150 cycles	200 cycles	250 cycles	300 cycles	
A	I	94	101	100	99	92	86	81	67	67	99	100	96	92	88	75	64	--------	53
A	IS	99	104	102	106	105	102	95	86	86	101	102	101	104	102	90	81	75	75
B	I	97	104	99	100	93	87	81	67	67	100	100	96	93	86	67	61	--------	51
B	IS	99	105	103	106	106	105	99	89	89	101	104	103	103	99	82	64	60	60
C	I	99	106	96	94	81	72	61	--------	51	100	101	96	91	87	72	53	--------	46
C	IS	101	105	103	102	102	99	96	86	86	100	102	101	98	94	79	73	66	66
D	I	100	104	102	98	94	87	81	70	70	101	103	96	94	92	82	64	--------	53
D	IS	101	104	103	103	104	102	98	92	92	98	98	100	98	95	83	74	66	66
E	I	102	104	101	102	94	86	79	65	65	99	100	96	93	92	74	60	--------	50
E	IS	98	102	101	105	104	100	94	88	88	101	102	100	98	92	78	69	63	63
Average, type I										64									51
Average, type IS										88									66

[1] Specimens were 3- by 4- by 16-inch beams frozen and thawed in accordance with ASTM Method C–292 for slow freezing and thawing in water. Each value is an average of tests on 5 beams.

test when 1 percent of opal was used, that mortar prepared with the slag cement with the greatest amount of alkali did not develop the most expansion at an age of 1 year. Apparently, some component of the cement had a modifying influence on the amount of expansion. It was believed that this might have been the amount of slag in the slag cement. It was learned from the manufacturers that the slag cements from sources A and B contained 45 percent slag, that from source C had 25 percent, that from source D had 40 percent, and that from source E had 35 percent.

In order to take this into account, figure 6 was prepared. This shows the relation between the amount of expansion of mortar containing 1-percent opal and tested at an age of 1 year, and a value obtained by dividing the slag content of each cement by the equivalent alkali content expressed as sodium oxide. The curve in the figure shows that the expansion decreases as the ratio of the slag content of the cement increases. It also shows that for equivalent alkali content of 0.6 percent, for example, it would be highly desirable to have the slag content of a slag cement appreciably higher than 30 percent, to prevent the alkali-aggregate reaction.

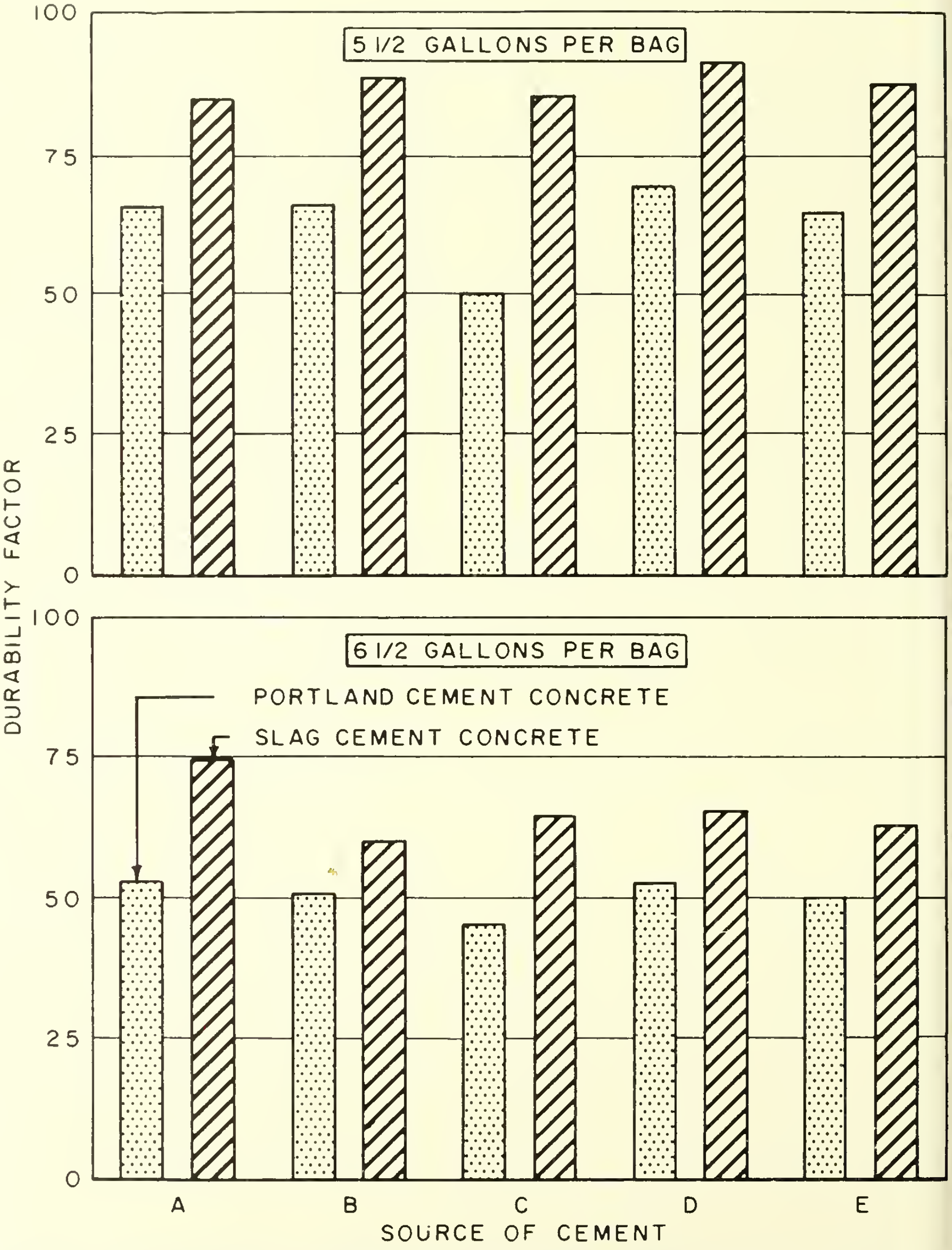

Figure 5.—Durability factors derived from freezing and thawing tests for portland and slag cement concretes.

Table 12.—Resistance of concrete to scaling[1]

Cement		Rating[2] after freezing and thawing with calcium chloride for indicated cycles								
Source	Type	5½ gallons per bag of cement			6½ gallons per bag of cement			7½ gallons per bag of cement		
		20 cycles	35 cycles	55 cycles	20 cycles	35 cycles	55 cycles	20 cycles	35 cycles	55 cycles
A	I	0	0	1	4	7	9	7	8	10
A	IS	0	1	1	4	7	9	4	9	9
B	I	1	2	2	4	7	9	6	8	9
B	IS	1	2	2	4	9	9	6	9	10
C	I	2	3	8	4	6	6	8	9	10
C	IS	2	6	7	6	9	9	6	9	10
D	I	1	2	4	8	9	10	9	9	10
D	IS	4	6	7	7	9	10	8	9	10
E	I	2	5	8	2	8	9	8	9	10
E	IS	4	6	7	6	9	9	9	9	10

[1] Specimens were 16- by 20- by 4-inch slabs, stored in moist air 28 days prior to placing in exposure area. Each value is an average of 2 tests. Air content approximately 5½ perceut.

[2] Rating of 0 indicates no scaling and 10 indicates deep scaling over entire surface.

Table 13.—Mortar bar expansion test results[1]

Cement				Expansion of 1- by 1- by 10-inch mortar bars, expressed in percent																	
Source	Type	Slag[2]	Alkali[3]	ASTM Method C 227-52 T			Federal SS-C-208B			Modified test: 0 percent opal			Modified test: 0.5 percent opal			Modified test: 1.0 percent opal			Modified test: 2.0 percent opal		
				8 wk.	6 mo.	1 yr.	8 wk.	6 mo.	1 yr.	8 wk.	6 mo.	1 yr.	8 wk.	6 mo.	1 yr.	8 wk.	6 mo.	1 yr.	8 wk.	6 mo.	1 yr.
A	I	--	0.32	0.12	0.19	0.20	0.02	0.05	0.03	0.02	0.02	0.02	0.12	0.28	0.35	0.17	0.33	0.56	0.07	0.10	0.13
A	IS	45	.37	.00	.00	.00	.00	.00	.01	.01	.01	.02	.02	.03	.03	.03	.08	.12	.04	.04	.07
B	I	--	.19	.02	.14	.16	.03	.07	.10	.03	.03	.04	.02	.06	.12	.03	.06	.06	.02	.03	.04
B	IS	45	.22	.00	.00	.01	.00	.00	.00	.02	.02	.02	.02	.03	.03	.01	.02	.03	.02	.02	.03
C	I	--	.32	.14	.23	.24	.10	.10	.14	.01	.02	.02	.05	.11	.18	.07	.16	.35	.04	.08	.10
C	IS	25	.61	.02	.07	.11	.00	.01	.02	.00	.01	.02	.18	.27	.27	.24	.40	.45	.16	.24	.32
D	I	--	.21	.00	.05	.10	.01	.02	.03	.01	.01	.02	.01	.02	.07	.00	.01	.02	.01	.01	.02
D	IS	40	.22	.00	.00	.00	.00	.00	.00	.02	.01	.01	.02	.01	.02	.02	.01	.01	.01	.01	.01
E	I	--	.82	.20	.26	.22	.16	.13	.16	.02	.02	.02	.12	.14	.14	.26	.29	.30	.38	.55	.57
E	IS	35	.75	.05	.06	.07	.04	.04	.04	.01	.01	.02	.08	.11	.11	.14	.26	.28	.10	.20	.26

[1] Tests were made at 8 weeks, 6 months, and 1 year. Each value is average of tests on 4 or 6 beams.

[2] Percent of slag used in manufacturing type IS cement.

[3] Equivalent alkalies as Na_2O (see table 1).

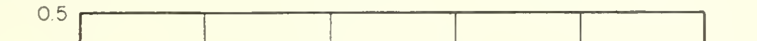

Figure 6.—Effect of slag content and alkali content of slag cements on expansion of mortar bars containing 1 percent of alkali-reactive opal (ratios expressed as percentages).

New Publications

Manual on Uniform Traffic Control Devices

The Bureau of Public Roads has just published the newly revised *Manual on Uniform Traffic Control Devices for Streets and Highways.* The new standards, updated from 1948, were drafted by the National Joint Committee on Uniform Traffic Control Devices, and approved by the Committee's member organizations, the American Association of State Highway Officials, the Institute of Traffic Engineers, the National Committee on Uniform Traffic Laws and Ordinances, the American Municipal Association, and the National Association of County Officials. The latter two organizations joined the National Joint Committee during the past year and are expected to impart added impetus to the modernization of traffic control devices throughout the nation.

The Bureau of Public Roads actively assisted the National Joint Committee in its work, and has a responsible interest in seeing that the results are broadly applied. By existing Federal highway legislation the signs, signals, and markings installed on highways constructed with Federal-aid funds are subject to approval by the State highway department with concurrence of the Federal Highway Administrator, who is directed to concur only in installations that promote safe, efficient highway use.

This new edition is expected to lend valuable service towards the safe and efficient use of the new highways being constructed in the Federal-aid program, as well as of the older streets and highways.

First published in 1935, and periodically reviewed and revised, the Manual reflects widely accepted and time-tested traffic control practices in the design and application of control devices, as well as extensive research into the principles of safe and orderly movement of vehicles and pedestrians. The newest edition includes, for the first time, specific standards for expressway signing, a major section on signing and marking for construction and maintenance operations, and a brief treatment of civil defense signing.

A significant feature of the new Manual is its elimination of certain alternatives in traffic control devices that previously were permitted, and the substitutions of a single standard. A notable example of this is the stripe to mark no-passing zones. In the future, according to the Manual, all such zones are to be marked with a yellow line to the right of the center stripe.

Another innovation is that, in general, the sizes of the newly specified traffic signs will be larger than those now in use, to provide greater visibility, particularly on multilane highways where driving is at higher speeds. Freeways and expressways in particular are to have larger and higher signs, and in specified places they will be placed overhead.

The *Manual on Uniform Traffic Control Devices for Streets and Highways* may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at $2 per copy.

Peak Rates of Runoff From Sma
Watersheds

Peak Rates of Runoff From Small Wate
sheds, published in April 1961, is No. 2 of tl
Bureau of Public Roads hydraulic desig
series. It reports a research study of pea
rates of runoff from small watersheds, ar
presents a procedure for the practical appl
cation of the results of that study. The stud
was limited to watersheds with areas of 2
square miles or less, located east of the 105t
meridian.

In parts I and II of the publication, stati
tical analyses of data from samples of gage
and ungaged watersheds demonstrate th
there is a correlation between a topograph
index, a precipitation index, and the wate
shed area; and a correction coefficient
developed for use when the topographic ind
indicates differences in drainage characteri
tics of the watersheds.

Parts III and IV present a practical pr
cedure for application of the research result
and include a discussion of some of the cor
siderations that must be taken into accour
in its use. The process, described step b
step, involves the use of lithological and rai
fall index maps and a series of correlatio
nomographs.

Peak Rates of Runoff From Small Wate
sheds is available from the Superintendent
Documents, U.S. Government Printing Offic
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U.S. GOVERNMENT PRINTING OFFICE: 1961

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VOL. 31, NO. 10

OCTOBER 1961

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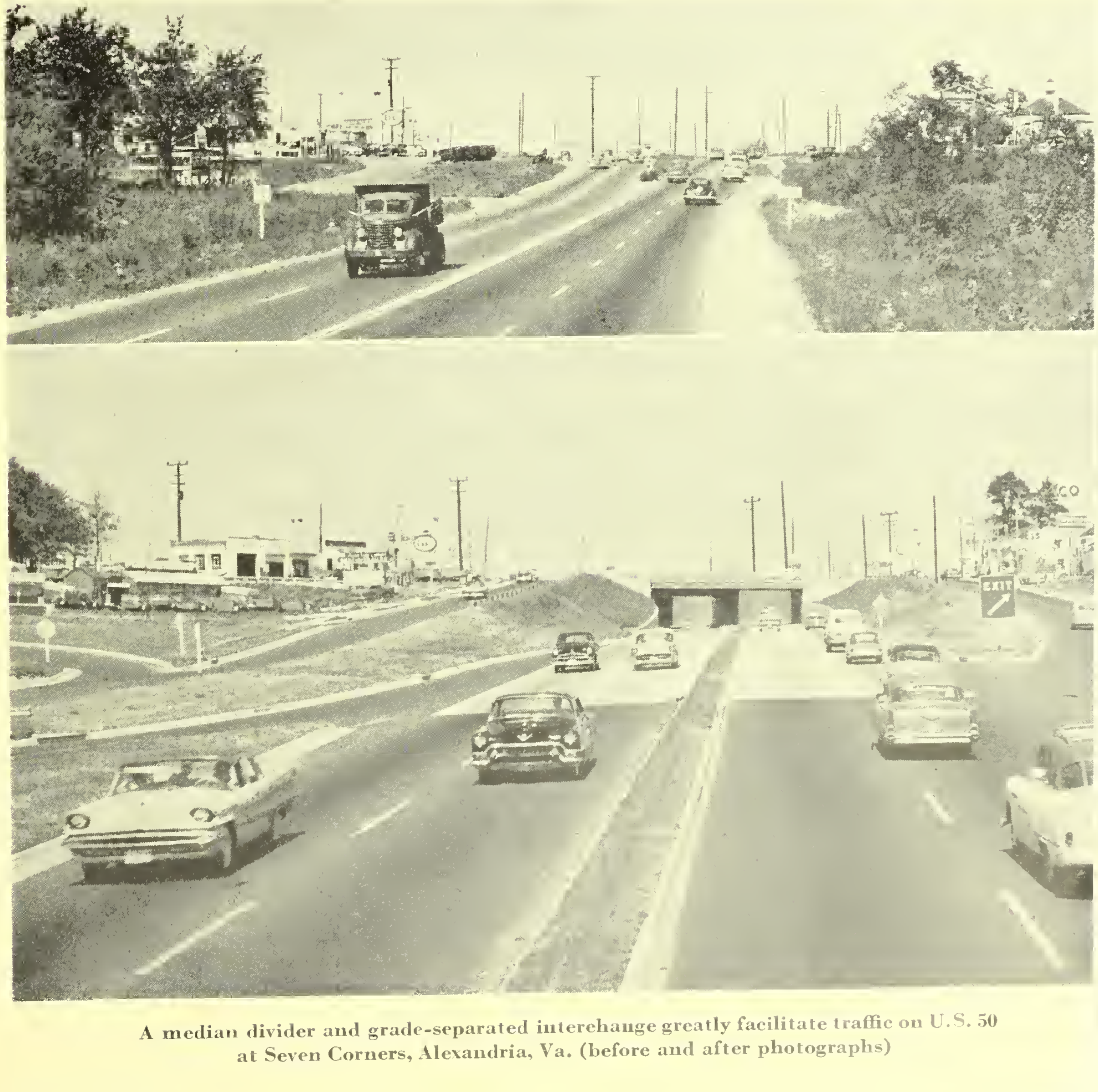

A median divider and grade-separated interchange greatly facilitate traffic on U.S. 50 at Seven Corners, Alexandria, Va. (before and after photographs)

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U.S. DEPARTMENT OF COMMERCE
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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 6, 1961.

Contents of this publication may be reprinted. Mention of source is requested.

A New Base for The Highway Construction Bid Price Index Compiled by the Bureau of Public Roads

BY THE CONSTRUCTION AND MAINTENANCE DIVISION
BUREAU OF PUBLIC ROADS

Reported by EDWIN L. STERN, Head,
Construction Costs and Price Trends Section

For almost 30 years the Bureau of Public Roads highway construction bid price index has been a widely accepted and frequently used measure of trends in the cost of highway construction. The index, computed from information on some 30 major bid items taken from awards of contracts for Federal-aid projects, has been published quarterly since 1933. Based on the 1925–29 period, it had been extended back as far as 1922.

Many changes have occurred in the nature of highway construction since the pattern for computing the index was first developed: quantities per mile differ radically; geographic distribution of work included in the base period was much different than it is now; and bituminous pavement was not directly included in the index.

In order to overcome such deficiencies, a new construction bid price index has now been devised, using 1957–59 as a base. Surfacing will now be represented by both portland cement concrete and bituminous concrete as indicators, and nationwide total quantities will be used in lieu of per-mile quantities for the base period.

The new index will be used exclusively beginning with the report for the first quarter of 1962 and, to provide transition in use, both the new and old indexes will be included in the reports for the third and fourth quarters of 1961. The design of the new index is explained in this article, and the historical trends from 1922 are shown on the new 1957–59 base.

Introduction

IN HIGHWAY CONSTRUCTION, as in many other undertakings, the need for future planning is readily apparent; and planning for the future can best be accomplished if highway engineers and economists have a thorough knowledge of what has taken place in the past. In order that physical volumes of work required for needed highway improvements may be expressed in terms of dollars required to purchase that work, high-level planning for future highway construction requires a knowledge of the amount of highway improvement that can be purchased by any given appropriation, at current prices. This, in turn, requires a knowledge of the changes and trends in average bid prices of highway construction items over past years.

To provide definite information as to price movements and trends in highway construction, the Bureau of Public Roads computes and publishes a continuing record of quarterly variations in contract bid prices on Federal-aid highway construction. This price trend record extends back to 1922, when highway construction in the United States had reached sufficient nationwide volume and had become sufficiently standardized to provide a fairly reliable basis for the comparison of unit construction costs and progressive price movements.

The basic data for the index, compiled in the Bureau of Public Roads field offices, are obtained from the awards of contracts for Federal-aid highway projects by the State highway departments, and consist of quantities, contract unit prices, and costs for approximately 30 major bid items. These data are reported to the Public Roads Washington Office for analysis, compilation, and summarization on a nationwide basis. The resultant index has been published quarterly since 1933 as *Price Trends for Federal-aid Highway Construction* (often referred to as the "composite mile index"), and has been widely recognized in the highway construction field as a standard measure of changes and indicator of trends in highway construction bid prices.

Basis of 1925–29 Index

The details of the design of the present price index are described in an article, *An Index of the Cost of Highway Construction*, published in July 1933 in Public Roads (vol. 14, No. 5). Briefly stated, this fixed-base type index was designed to show what it would cost, at any given time, to contract for the Federal-aid construction put in place during the 1925–29 base period. Strictly speaking, a composite highway price index should be a weighted composite of over 100 items, but the computation of such an index would involve an extremely large amount of work. Fortunately, the nature of highway construction is such that changes in prices of certain principal work items are generally accompanied by similar changes in prices of associated items or classes of work.

For example, in preparing the index on the 1925–29 base, it was assumed, with reason, that if the unit price of common excavation increased 5 percent during a quarterly period, the prices of all other types of excavation also increased approximately 5 percent during the same period. Likewise, it was assumed that percentage changes in prices of all high-type surfacing would be similar to percentage changes in portland cement concrete pavement prices. For this reason, common excavation and concrete pavement were used as indicator items for grading and surfacing, respectively. For structures, three indicator items were used—reinforcing steel, structural steel, and structural concrete.

The relative weights for the index items were determined from expenditures for the various construction items during the base period by grouping together those items represented by each indicator item. For example, all expenditures for excavation and low-type bases were combined, and the equivalent quantity of common excavation that could be contracted for by this expenditure was derived by dividing the combined expenditure by the base-period weighted average price per cubic yard of common excavation. This latter figure, the weighted average price for common excavation, was computed by dividing the total contract quantity of common excavation in all of the States for the years 1925–29 into the total contract dollar amount for this excavation.

Similarly, the equivalent area of portland cement concrete paving was derived by dividing the sum of all money spent for high-type surfacing by the base-period weighted average price per square yard for concrete paving. No such conversion was necessary for structures, since the costs for the three index items—reinforcing steel, structural steel, and structural concrete—constituted the bulk of structure costs.

The five base-period quantities—for excavation, paving, reinforcing steel, structural steel, and structural concrete—thus obtained were each reduced to a relative quantity-per-mile basis by dividing each by the total mileage of construction during the base period. (This total mileage represented construction of various kinds, of course.) The composite index of highway construction cost for any subsequent period could then be computed by (1) multiplying these base-period per-mile quantities for the five items by the applicable current average unit price for each item, (2) dividing the sum of these products by the sum of the products of the base-period per-mile quantities and the base-period average unit prices, and (3) multiplying the quotient by 100.

This highway construction bid price index is known as a fixed-base index. It provides a means for determining what it would cost to contract for the base-period quantities at the current prices of any particular period. In other words, the cost of the base-period quantities at prices of a particular period are compared with the cost of the base-period quantities at the base-period prices.

The reliability of the trends in the fixed-base index has been checked periodically by the computation of a changing-base index, in which the quantities for the particular period are used instead of the quantities for the 1925–29 base period. In addition, as a further check, the changing-base index computations employ the quantities for all major highway construction bid items, in lieu of the quantities for only the five indicator items. This changing-base index provides a means for determining how much more or less it would cost to contract for particular-period quantities at particular-period prices than it would cost to contract for the same quantities at base-period prices. It has been found that, over a period of years, the trends developed by the fixed-base method produce considerably smoother curves than the changing-base method. This better stability of the fixed-base index indicates its greater reliability for use in the prediction of future price trends.

Deficiencies in the 1925–29 Index

Many changes have occurred in the nature of highway construction during the three decades since the fixed-base index was first developed. As a consequence, certain deficiencies have become apparent in recent years, as described in the following paragraphs.

Table 1.—Variation of percentage values of price index indicator items with use of 1925–29 and 1957–59 base data

Item	1925–29 quantities at 1925–29 prices	1925–29 quantities at 1957–59 prices	1957–59 quantities at 1957–59 prices
	Percent	*Percent*	*Percent*
Excavation	36	24	34
Surfacing:			
Portland cement concrete	48	54	15
Bituminous concrete	0	0	16
Subtotal, surfaces	48	54	31
Structures:			
Reinforcing steel	5	7	6
Structural steel	2	3	11
Structural concrete	9	12	18
Subtotal, structures	16	22	35
Total	100	100	100

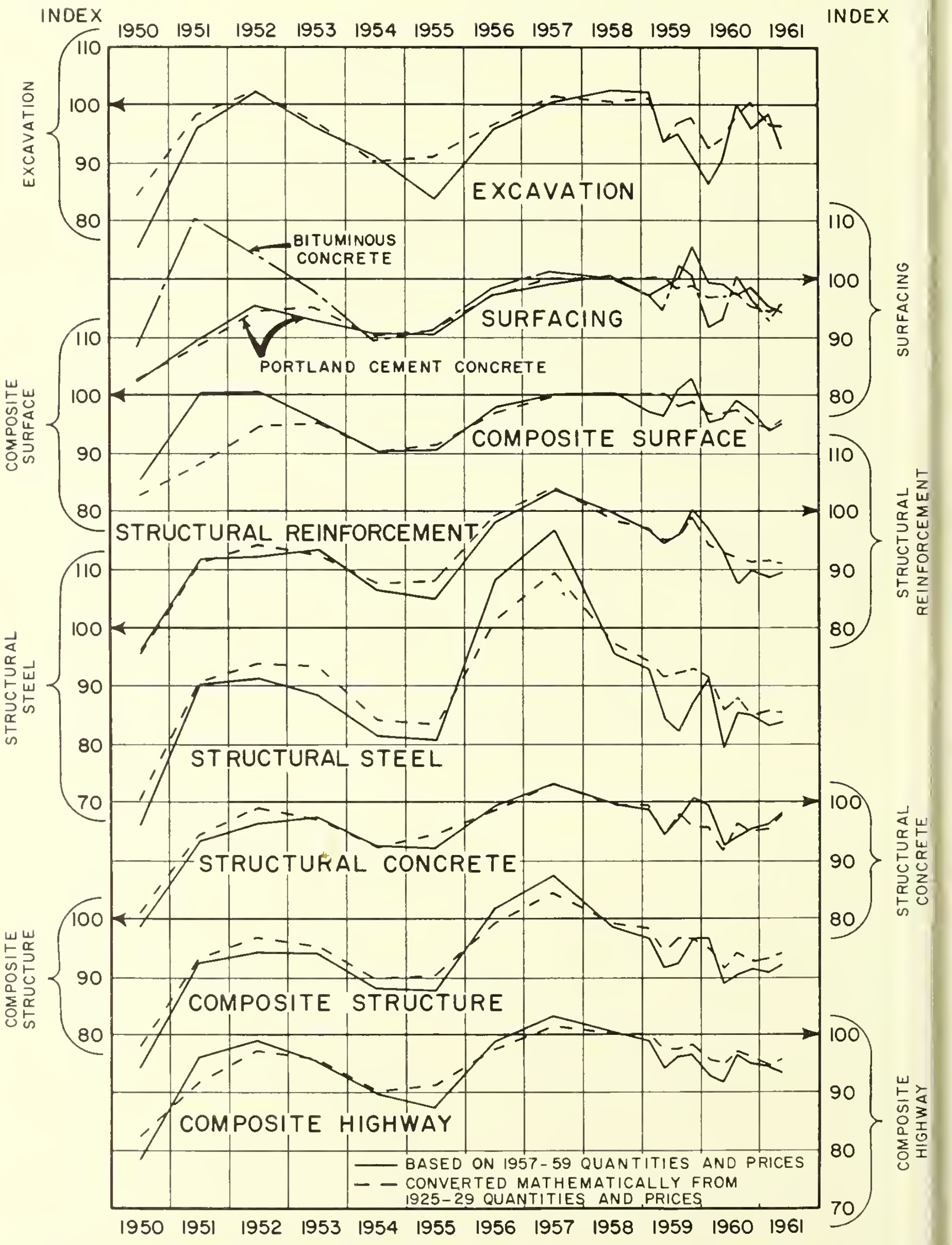

Figure 1.—Comparison of trends of the original and new bid price indexes, 1950 to date.

Quantities.—The 1925–29 base period is quite remote and the base-period per-mile quantities are no longer representative of present Federal-aid highway work. While it is true that the magnitudes of the base quantities do not affect the computed index values so long as their magnitudes relative to each other do not change, there has been a considerable change in the relative weights of grading, surfacing, and structures from those of the base period. Table 1 shows a comparison of the percentage values for the indicator items as computed for the 1925–29 quantities at 1925–29 prices and at 1957–59 prices and for 1957–59 quantities at 1957–59 prices. Although the actual quantity of surfacing put in place during the 1957–59 period was much greater than that put in place during the 1925–29 period, percentagewise surfacing decreased considerably in relation to grading and structures. This results principally because of the higher level of geometric standards adopted for highway construction during the past two decades, requiring proportionally more earthwork and structures.

Geographic distribution.—The geographic distribution of the work during the base period 1925–29 was quite different from 1957–59 or present-day periods. This has an appreciable effect, since quantities and bid prices vary considerably across the country.

Bituminous pavements.—The continued calculation of the composite mile index without direct consideration of the costs of bituminous pavements created some doubt as to its accuracy, especially in view of the heavy relative weight of surfacing in the composite index. As previously stated, the basic assumption in the original design of the index was that prices for portland cement concrete pavement would accurately reflect the price movements for all types of pavements. This was probably valid at the time. It has been determined that there is some similarity in the price trends of portland cement concrete pavement and bituminous concrete pavement (the bituminous type chosen as an indicator for the redesigned index). However, a difference of several percent in the index results when the actual average prices for each type are separately derived.

Index Redesigned

In order to overcome these deficiencies, the original fixed-base index has now been redesigned. The new index will employ the 3-year period 1957–59 as a base, in accordance with the general endeavor of the Federal Government, under the leadership of the Bureau of the Budget, to establish all indexes published by Federal agencies on the uniform 1957–59 base period, beginning January 1, 1962. The method of computation for the new 1957–59 base index is similar to that used for the 1925–29 base index except that sur-

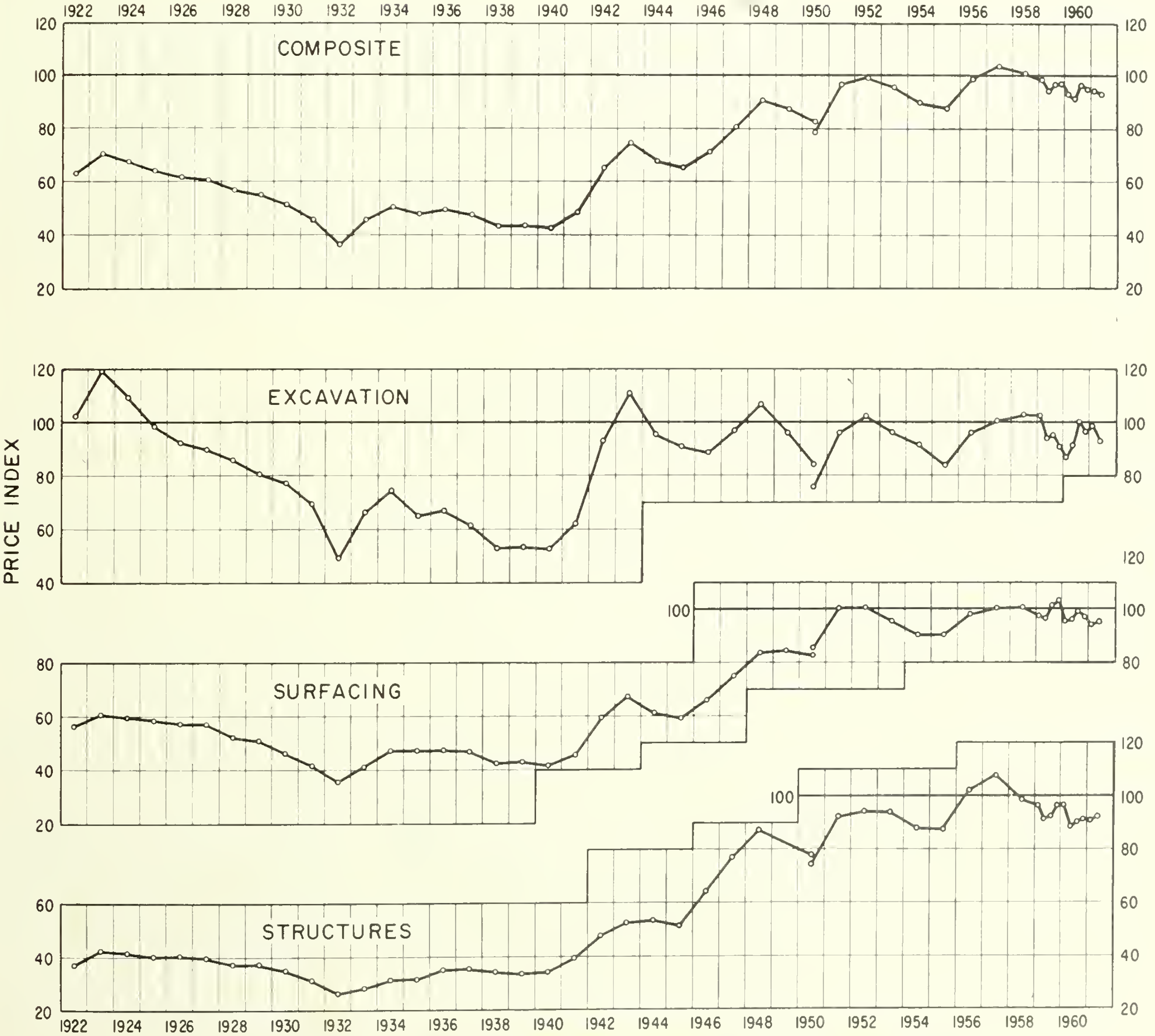

Figure 2.—Trends of the new bid price indexes, 1922 to date. (See explanation in text.)

facing will be represented by two indicator items, portland cement concrete and bituminous concrete. In addition, nationwide total quantities will be used in lieu of per-mile quantities for the base period. The method for computing the 1957–59 total base quantities of the five original items is the same as described for the 1925–29 total base quantities. The 1957–59 bituminous concrete surfacing base quantity, in tons, was computed by dividing the 1957–59 United States average unit price per ton for bituminous concrete surfacing into the total 1957–59 contract cost for all types of bituminous surfacing.

Figure 1 shows comparisons, from 1950 to date, of the trends of the original index, based on 1925–29 quantities but converted mathematically to a 1957–59 base = 100, and the trends of the new index based on 1957–59 quantities. (The trend of bituminous surfacing prices is shown only for 1957–59 quantities, since this element was not included in the 1925–29 base index.) The variations are significant in some periods.

Table 2 shows the 1957–59 base quantities, average unit prices, total dollar amounts, and relative weights of the index items. As already shown in table 1, the relative weights in the new index vary considerably from those in the old index.

Table 2.—Basic values and relative weights of the 1957–59 base index

Item	Unit	Base quantity	Base unit price	Base dollar amount	Relative weight
		Thousands		*Thousands*	*Percent*
Excavation	cu. yd	3, 641, 885	$0. 42	$1, 529, 592	33. 8
Surfacing:					
Portland cement concrete	sq. yd	154, 953	4. 38	678, 221	15. 0
Bituminous concrete	ton	111, 516	6. 66	742, 472	16. 4
Subtotal, surfaces				1, 420, 693	31. 4
Structures:					
Reinforcing steel	lb	2, 206, 879	. 129	285, 139	6. 3
Structural steel	lb	2, 581, 462	. 195	502, 294	11. 1
Structural concrete	cu. yd	14, 583	54. 18	790, 027	17. 4
Subtotal, structures				1, 577, 460	34. 8
Total				4, 527, 745	100. 0

In developing the 1957–59 based index for any given period, the index for each item is computed by dividing the base-period unit price into the United States average unit price for the particular period, and multiplying the quotient by 100. The composite surfacing index is computed by multiplying the average unit prices of the given period for portland cement concrete surfacing and bituminous concrete surfacing by their respective base quantities, dividing the sum of these products by the total base-period amount ($1,420,693,000, from table 2) for the two items, and multiplying the quotient by 100. The composite structure index, and the composite highway construction cost index (for excavation, surfacing, and structures combined), are computed in a similar manner, using the appropriate prices, quantities, and dollar amounts.

Figure 2 is the graphical representation of the new index trends; table 3 (p. 199) shows the data on which the curves are based. Index figures for 1922–50 are simple mathematical conversions from the 1925–29 base to the 1957–59 base. They were derived from the previously computed figures, using 1925–29 base quantities and prices, and dividing the figures for each year by the average of the figures for the years 1957, 1958, and 1959. Revised figures for 1950 and subsequent years are computed from 1957–59 base quantities and prices. The breaks in the curves in figure 2 indicate indexes for 1950 computed by both methods. Prices for portland cement concrete surfacing reflect adjustments to base period thicknesses in each State and do not include costs for reinforcing steel and joints.

The new index, computed on the 1957–59 base, will be used beginning with the report for the first quarter of calendar year 1962. In order to facilitate the change-over, the reports of price trends for Federal-aid highway construction for the third and fourth quarters of calendar year 1961 will include index data on both the 1925–29 and 1957–59 bases.

New Publications

Landslide Investigations

The Bureau of Public Roads has recently published *Landslide Investigations*, a field handbook on landslides and related phenomena. Prepared primarily for the highway location engineer, the publication should also be of interest to other engineers, geologists, teachers, and students.

The convenient pocket-size, 67-page handbook is divided into four parts. The brief introductory section describes geologic processes, rocks and soil types, and geologic structures which provide the setting for landslides. The second section involves the recognition of phenomena presaging the advent of slide movements, and those characteristic of the landslide itself. The third is devoted to methods of landslide prevention, control, and correction. The fourth section discusses details of mapping and reporting landslides. A glossary of terms is also included.

The 31 illustrations, with self-explanatory captions, portray both conventional techniques and some unique methods for overcoming landslide problems. The handbook, designed for field use and not intended to be an exhaustive treatise, lists five major publications on the subject of landslides.

Landslide Investigations is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 30 cents per copy.

Design Charts for Open-Channel Flow

The Bureau of Public Roads has recently published *Design Charts for Open-Channel Flow*, making generally available a group of hydraulic charts which facilitate the computation of uniform flow in open channels. Some of the charts are also useful in the design of storm drains. The publication is not intended to be a treatise on the design of open channels, although a brief discussion of the principles of flow in open channels is included. Rather, it is intended as a working tool that should be of considerable service to the designer already familiar with the subject.

The 105-page publication contains 82 charts which provide direct solution of the Manning equation for uniform flow in open prismatic channels of various cross sections; instructions for using the charts; a table of recommended values of n in the Manning equation and tables of permissible velocities in earth and vegetated channels; instructions for constructing charts similar to those presented; and a nomograph for use in the solution of the Manning equation. Charts are included for rectangular, trapezoidal, and triangular channels, grass-lined channels, circular pipe channels, pipe-arch channels, and oval concrete pipe channels.

Practically all of the material in this publication has appeared previously in the processed *Highway Drainage Manual* prepared by the Public Roads Regional Office in Hagerstown, Md.

Design Charts for Open-Channel Flow, which is the third in a series on the hydraulic design of highway drainage structures published by the Bureau of Public Roads, is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 70 cents per copy.

Table 3.—Price trends for Federal-aid highway construction, 1957–59 base [1]

Year	Common excavation		Surfacing					Structures							Composite index
			Portland cement concrete		Bituminous concrete		Surfacing index	Reinforcing steel		Structural steel		Structural concrete		Structures index	
	Bid price	Index	Bid price	Index	Bid price	Index		Bid price	Index	Bid price	Index	Bid price	Index		
INDEXES CONVERTED MATHEMATICALLY FROM 1925–29 BASE TO 1957–59 BASE [2]															
	Cu. yd.		*Sq. yd.*		*Tons*			*Lb.*		*Lb.*		*Cu. yd.*			
1922	$0.40	102.5	$2.28	56.9	------	------	56.9	$0.050	38.1	$0.074	37.3	$20.18	36.6	37.2	63.2
1923	.47	119.3	2.43	60.6	------	------	60.6	.057	43.4	.078	39.4	23.37	42.4	42.5	70.4
1924	.43	109.4	2.40	59.9	------	------	59.9	.057	43.4	.077	38.8	22.91	41.6	41.9	67.6
1925	.39	98.5	2.36	58.9	------	------	58.9	.056	42.6	.067	33.8	22.53	40.9	40.6	64.2
1926	.36	92.8	2.29	57.1	------	------	57.1	.053	40.3	.074	37.3	22.76	41.3	40.6	61.8
1927	.35	89.8	2.29	57.1	------	------	57.1	.051	38.8	.071	35.8	22.65	41.1	39.7	60.9
1928	.34	85.9	2.10	52.4	------	------	52.4	.049	37.3	.067	33.8	21.22	38.5	37.6	56.9
1929	.32	80.6	2.05	51.1	------	------	51.1	.048	36.5	.059	29.8	21.58	39.2	37.2	55.0
1930	.30	77.3	1.86	46.4	------	------	46.4	.045	34.2	.061	30.8	20.08	36.4	35.0	51.2
1931	.27	69.5	1.68	41.9	------	------	41.9	.040	30.4	.054	27.2	18.02	32.7	31.3	45.9
1932	.18	49.2	1.44	35.9	------	------	35.9	.034	25.9	.046	23.2	15.33	27.8	26.6	36.4
1933	.26	66.2	1.67	41.6	------	------	41.6	.038	28.9	.046	23.2	16.15	29.3	28.2	45.8
1934	.29	74.6	1.90	47.4	------	------	47.4	.043	32.7	.053	26.7	17.73	32.2	31.6	50.2
1935	.26	65.6	1.90	47.4	------	------	47.4	.044	33.5	.052	26.2	17.78	32.3	31.9	48.1
1936	.26	67.0	1.91	47.6	------	------	47.6	.046	35.0	.060	30.3	20.25	36.8	35.2	49.5
1937	.24	61.8	1.89	47.1	------	------	47.1	.048	36.5	.066	33.3	19.76	35.9	35.8	47.4
1938	.21	53.5	1.72	42.9	------	------	42.9	.045	34.2	.063	31.8	19.06	34.6	34.1	43.5
1939	.21	53.5	1.73	43.1	------	------	43.1	.044	33.5	.059	29.8	19.13	34.7	33.6	43.4
1940	.21	53.0	1.68	41.9	------	------	41.9	.045	34.2	.063	31.8	19.17	34.8	34.3	42.8
1941	.24	62.0	1.87	46.6	------	------	46.6	.054	41.1	.076	38.3	21.44	38.9	39.4	48.6
1942	.37	93.2	2.39	59.6	------	------	59.6	.065	49.5	.090	45.4	26.16	47.5	47.8	65.0
1943	.44	111.0	2.71	67.6	------	------	67.6	.067	51.0	.095	47.9	30.19	54.8	52.6	74.6
1944	.37	95.4	2.45	61.1	------	------	61.1	.064	48.7	.089	44.9	31.94	58.0	53.4	67.6
1945	.36	91.0	2.38	59.4	------	------	59.4	.062	47.2	.077	38.8	31.62	57.4	51.8	65.1
1946	.35	88.7	2.65	66.1	------	------	66.1	.075	57.1	.113	57.0	38.79	70.4	64.4	71.1
1947	.38	97.0	3.01	75.1	------	------	75.1	.093	70.8	.132	66.6	45.84	83.2	77.0	80.6
1948	.42	106.8	3.37	84.0	------	------	84.0	.108	82.2	.158	79.7	51.00	92.6	87.6	90.3
1949	.38	96.2	3.40	84.8	------	------	84.8	.104	79.1	.146	73.7	47.36	86.0	82.3	87.1
1950	.33	84.6	3.32	82.8	------	------	82.8	.100	76.1	.139	70.1	44 62	81.0	78.0	82.3
INDEXES COMPUTED FROM 1957–59 BASE QUANTITIES AND PRICES [3]															
1950	$0.32	75.7	$3.62	82.7	$5.89	88.5	85.7	$0.099	76.2	$0.129	66.1	$42.62	78.7	74.2	78.3
1951	.40	96.2	3.92	89.6	7.33	110.1	100.3	.119	92.0	.176	90.4	50.72	93.6	92.3	96.1
1952	.43	102.4	4.19	95.7	6.98	104.8	100.5	.119	92.4	.178	91.3	52.24	96.4	94.1	98.9
1953	.40	96.2	4.07	93.0	6.53	98.1	95.6	.121	93.8	.172	88.6	52.82	97.5	94.0	95.3
1954	.38	91.4	3.98	90.9	5.97	89.7	90.3	.112	86.7	.159	81.5	50.15	92.6	88.0	89.9
1955	.35	84.0	3.96	90.5	6.07	91.2	90.8	.110	85.2	.157	80.9	50.01	92.3	87.4	87.3
1956	.40	96.0	4.26	97.3	6.58	98.8	98.1	.127	97.9	.212	108.9	53.74	99.2	102.0	98.8
1957	.42	100.7	4.34	99.2	6.75	101.4	100.3	.134	104.0	.228	117.0	55.98	103.3	107.8	103.1
1958	.43	102.9	4.41	100.8	6.67	100.2	100.5	.129	99.9	.186	95.7	54.10	99.9	98.5	100.6
1959:															
1st quarter	.43	102.6	4.26	97.3	6.50	97.6	97.5	.125	97.1	.181	93.1	53.53	98.8	96.7	98.9
2nd quarter	.40	94.0	4.33	98.9	6.31	94.8	96.8	.123	94.9	.165	84.6	51.19	94.5	91.4	94.0
3rd quarter	.40	95.2	4.38	100.1	6.82	102.4	101.3	.124	96.3	.160	82.3	52.59	97.1	92.2	96.1
4th quarter	.38	90.7	4.64	106.0	6.70	100.6	103.2	.130	100.7	.170	87.1	54.67	100.9	96.5	96.6
Average	.40	95.7	4.40	100.5	6.58	98.8	99.6	.126	97.2	.169	86.8	53.00	97.8	94.2	96.4
1960:															
1st quarter	.37	86.9	4.36	99.6	6.11	91.8	95.5	.125	97.0	.178	91.6	53.98	99.6	96.6	93.0
2nd quarter	.38	91.2	4.35	99.4	6.20	93.1	96.1	.121	93.3	.155	79.7	50.20	92.7	88.7	91.9
3rd quarter	.42	100.2	4.27	97.6	6.71	100.8	99.2	.114	87.8	.167	85.7	50.97	94.1	90.3	96.5
4th quarter	.41	96.4	4.33	98.9	6.44	96.7	97.8	.116	90.1	.166	85.1	51.71	95.5	91.2	95.0
Average	.39	93.8	4.33	98.9	6.37	95.7	97.2	.119	92.1	.167	85.6	51.72	95.5	91.7	94.1
1961:															
1st quarter	.41	98.3	4.18	95.5	6.19	93.0	94.2	.115	89.0	.162	83.4	52.14	96.2	90.8	91.4
2nd quarter	.39	92.6	4.13	94.4	6.38	95.8	95.1	.116	89.6	.163	83.9	53.25	98.3	92.2	93.2
1957–59 base [4]	0.42	100.0	4.38	100.0	6.66	100.0	100.0	0.129	100.0	0.195	100.0	54.18	100.0	100.0	100.0
INDEXES COMPUTED ON PREVIOUS QUARTER BASE [5]															
1960:															
1st quarter	------	95.8	------	94.0	------	91.2	92.6	------	96.3	------	105.2	------	98.7	100.1	96.2
2nd quarter	------	104.9	------	99.8	------	101.5	100.6	------	96.2	------	87.0	------	93.0	91.8	98.8
3rd quarter	------	109.9	------	98.2	------	108.2	103.3	------	94.1	------	107.5	------	101.5	101.8	105.0
4th quarter	------	96.2	------	101.4	------	96.0	98.5	------	102.6	------	99.3	----	101.5	101.0	98.5
1961:															
1st quarter	------	102.0	------	96.5	------	96.1	96.3	------	98.8	------	98.0	------	100.8	99.6	99.4
2nd quarter	------	94.2	------	98.8	------	103.0	101.0	------	100.7	------	100.6	------	102.2	101.5	98.7

[1] Base for composite index, 1957–59, involves 3,641,885,000 cu. yd. of roadway excavation; 154,953,000 sq. yd. of portland cement concrete surfacing with average thickness of 9.1 in.; 111,516,000 tons of bituminous concrete surfacing; 2,206,879,000 lb. of reinforcing steel for structures; 2,581,462,000 lb. of structural steel; and 14,583,000 cu. yd. of structural concrete.

[2] Derived from the previously computed figures, using 1925–29 base quantities and prices, and dividing the figures for each year by the average of the figures for 1957, 1958, and 1959.

[3] Prices for portland cement concrete surfacing reflect adjustments to base period thicknesses in each State and do not include costs for reinforcing steel and joints.

[4] Bid prices for the base period, 1957–59, for which the index is 100.

[5] Index for each quarter as computed using an index of 100 for the preceding quarter.

Dynamic Weighing of Vehicles

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A dependable method of weighing the axle loads of a moving vehicle is needed in the collection of highway planning data and as an aid to the enforcement of load limit regulations. A research project now under way at the University of Kentucky has as its objective the development of such a method. The first phase of the project included an appraisal of all known previous work in the field of dynamic weighing. This article reviews the development of the dynamic electronic scale and the experiences of a number of State highway departments with its practical use. Recent experimental developments in this country and Europe are described and evaluated. Some conclusions are reached and the future course of the Kentucky project is outlined.

Introduction

WHAT IS THE BEST and most practical method of weighing a moving vehicle? Should new equipment be developed for this purpose or can existing devices be modified to do the job? The answer to these and other questions on the subject are being sought by a research team at the University of Kentucky's College of Engineering.[1]

Working under joint sponsorship of the Kentucky Department of Highways and the Bureau of Public Roads, the team is conducting its investigations along these general lines:

An examination and appraisal of all known types of commercial and experimental dynamic weighing systems.

The design and construction of a test installation on an Interstate System route.

An exhaustive testing program, utilizing various types of dynamic scales and allied equipment.

Analysis of the test results, with the objective of developing specifications for a model dynamic weighing station.

The first step, collection of all available information on the subject of dynamic weighing, has been completed. It is realized, however, that some unpublished or unpublicized work may well have been missed by the research staff, and information on any such additional material for study would be appreciated.

After study of the available literature, a number of inspection trips were made to existing installations. This article summarizes the findings from both approaches, draws some conclusions, and outlines the future course of the project.

Purposes of Dynamic Weighing

The problem of weighing moving vehicles has received the attention of many agencies and individuals concerned with the planning and operation of our highway systems. A knowledge of the magnitude and frequency of application of wheel loads to the highway surface is of major importance in the structural design of the pavement cross section, and in anticipating future maintenance problems. This information is generally obtained at present by stopping a small sample of the passing vehicles and measuring the wheel loads with a portable scale or loadometer. The cost of weighing a 200-vehicle sample by this method has been estimated at about $150 per day (*1*)[2]. Obviously, an efficient dynamic weighing system in continuous operation would permit the collection of data for a much larger sample at a greatly reduced operating cost.

The trucking industry and the load limit enforcement agencies of the several States are also interested in the development of a dynamic scale that will determine axle weights of vehicles in the traffic stream with an accuracy sufficient to permit detection of overloads. Those trucks loaded near or above the legal limit could then be stopped for check weighing at a conventional static scale, if necessary for legal reasons. Empty or lightly loaded vehicles could proceed with little or no delay. The importance of this is emphasized by the fact that during one 14-week period in Illinois, over a million vehicles were stopped unnecessarily for weighing in connection with load-limit enforcement. Only 0.2 percent of the vehicles weighed were overloaded (*1*).

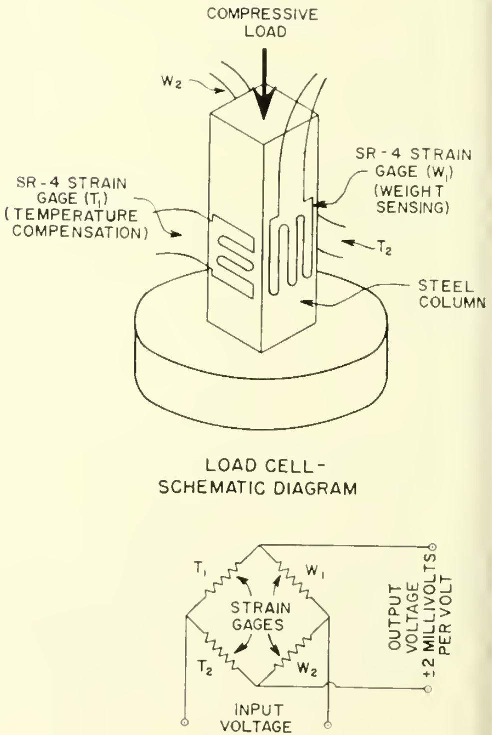
The Electronic Scale

The most widely used dynamic weighing device is the electronic scale. This type of scale has been used by industry for over a decade in the static weighing of such diverse items as steel, coal, cement, and aircraft; and for both dynamic and static weighing of railroad freight cars (*1*). The electronic scale will measure static weights with an accuracy equal to or better than that of the lever system scale. It is also light in weight and contains no levers, knife edges, or other mechanical parts requiring periodic adjustments due to wear.

As shown in figure 1, the basic weight sensing element of the electronic scale is the resistance wire strain gage, which is an encased fine wire grid mounted on a paper backing and covered with felt. When a load is applied to a structural member, to which the strain gage is cemented, the wire in the gage is lengthened or shortened a minute amount. This change in length results in a measurable change in the resistance of the wire, which is proportional to the applied load.

An important application of this principle for weighing purposes is in the commercially developed "load cell." The load cell, illustrated in figure 1, consists primarily of a short steel column to which are bonded resistance wire strain gages. The gages are so arranged that in each cell they form a resistance network commonly known as a Wheatstone Bridge. When the network is balanced, the voltage output of the cell is zero. When a load is applied to the cell the bridge becomes electrically unbalanced, resulting in a tiny voltage output which can be amplified and translated into an indication of weight.

The typical electronic scale consists of a rectangular platform supported at each of its corners by a load cell. The type of detecting and recording apparatus used depends upon the object being weighed and the kind of output


COMPRESSIVE LOAD
W_2
SR-4 STRAIN GAGE (W_1) (WEIGHT SENSING)
SR-4 STRAIN GAGE (T_1) (TEMPERATURE COMPENSATION)
T_2
STEEL COLUMN
T_1
W_1
STRAIN GAGES
T_2
W_2
OUTPUT VOLTAGE ±2 MILLIVOLTS PER VOLT
INPUT VOLTAGE (4-6 VOLTS)

LOAD CELL-SCHEMATIC DIAGRAM

TYPICAL LOAD CELL CIRCUIT

Figure 1.—The strain gage load cell.

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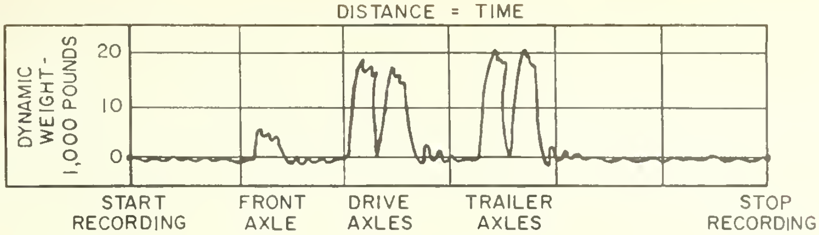
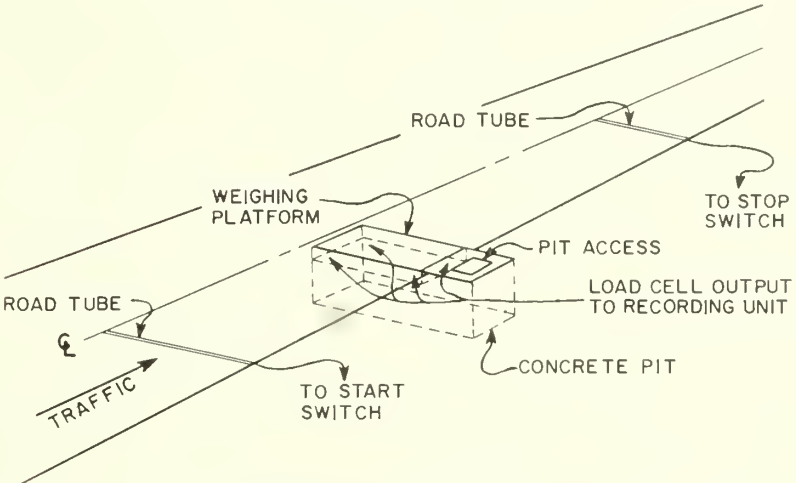
[1] Members of the research team are David K. Blythe, Civil Engineering Department Head, project director; John A. Dearinger, Assistant Professor of Civil Engineering, assistant project director; Russell E. Puckett, Assistant Professor of Electrical Engineering, electronics engineer; and Charles C. Schimpeler, graduate student in civil engineering, research assistant.

[2] Italic numbers in parentheses represent references, page 210.

desired. The scales for weighing cement and livestock, for example, use a printed card output (*2*). Most of the scales used for dynamic weighing are equipped with a recording oscillograph which records a weight as the peak reading of a stylus trace on paper tape. A typical oscillograph record is shown in figure 2.



TYPICAL OSCILLOGRAPH RECORDING
(5-AXLE TRUCK-TRAILER)



DYNAMIC SCALE INSTALLATION

Figure 2.—Highway installation of a dynamic weighing scale.

Development of Dynamic Weighing

The pioneer effort in the development of a dynamic weighing device for highway use was made by O. K. Normann and R. C. Hopkins of the Bureau of Public Roads, who, in 1951, installed an experimental scale in the Shirley Highway near Washington, D.C. (*3*).

This scale consisted of a 3-foot long (in the direction of traffic) by 10-foot wide concrete platform, supported on four load cells (see fig. 2). The electronic apparatus used was part of a portable kit designed for the static weighing of aircraft. The scale was calibrated so that the axle weights of a vehicle passing over the platform were shown as individual peaks in an oscilloscope pattern. Recording was accomplished by photographing the pattern at the time of the vehicle's passage. A pair of air switches, actuated by road tubes, provided automatic starting and stopping of the equipment, and enabled speed and axle spacing to be determined.

Accuracy tests were conducted by comparing the static axle weights of a 2-axle test vehicle with the dynamic weights recorded by the electronic scale, at vehicle speeds ranging from 10 to 50 miles per hour. The average difference between the weight recorded and the actual static weight was about 5 percent, and seemed to be independent of speed. A much larger variance was noted for the second and third axles of a 3-axle vehicle; this was thought to be due to the vibrations set up in the platform by the first axle. Horizontal tie-bars and vertical pre-loading bolts added to the platform succeeded in reducing the size of these differences by about 50 percent.

Further testing of the scale resulted in a number of recommendations for improving its design and operation. These recommendations were incorporated into a second scale constructed near Woodbridge, Va., by the Virginia Department of Highways in cooperation with the Bureau of Public Roads and an electronic equipment manufacturer. Major changes in the design of this scale included lengthening the platform to 7 feet (in the direction of traffic) and the use of a recording oscillograph instead of the oscilloscope and camera. The Woodbridge scale was used by the Public Roads research staff in their search for the best method of measuring variations in the dynamic loading of the highway surface (*4*).

Test data from the original scale had shown that the dynamic weights recorded varied from the static weights in a random manner—the recorded dynamic weight of an axle might be greater than, less than, or equal to its static weight. This indicated that the vehicles were oscillating as they crossed the platform, and that the scale was actually sampling the applied pressure at random points in the vehicle's oscillation cycle. This was verified by measuring dynamic load variations through air pressure changes in the tires of a moving vehicle and comparing these variations to the simultaneous weight recordings of the scale.

It was also found that the number of cycles through which the vehicle oscillated as it crossed the 7-foot platform was dependent upon speed and pavement roughness. For speeds over 20 miles per hour, less than one-half of a complete cycle occurred while the vehicle was on the platform. Rough approach conditions tended to decrease the frequency of the oscillations with a corresponding increase in their amplitude. The reverse was true for smoother approach surfaces.

These findings were significant. Since the dynamic weights ranged above and below the static weights in an unbiased way, the differences or "errors" would tend to cancel out of the sum of a number of recorded axle loads. Gross tonnage information needed by highway planners could thus be obtained on the electronic scale with an accuracy equal to that of the slower and more costly loadometer method. It was also made clear that a smooth approach to the scale platform was a necessity if a closer agreement between static and dynamic weights was to be realized.

Dynamic Weighing in the United States

Encouraged by the results of the Bureau of Public Roads experimental work, a few electronic equipment firms began manufacturing scales for dynamic weighing. These scales were all of the same general design as the Public Roads prototype—a short rectangular platform supported by load cells.

With the advent of the commercially available scale, several State highway departments became interested in its application to their particular problems. First to attempt the practical use of the dynamic scale were the States of Iowa, Minnesota, and Oregon. The degree of success attained by these States in their dynamic weighing operations appears to have depended mainly upon the type of information they were trying to obtain.

It was generally agreed that the electronic scale might well be used for one or more of the following purposes:

Dynamic weighing to obtain weight data for research and planning.

Dynamic weighing to detect overloaded trucks.

Static weighing for enforcement of legal weight limits.

As will be seen, these three States, in the course of their experiences with the device, have investigated rather thoroughly the practical side of each of these possibilities. All of these States completed their initial installations at about the same time (in the fall of 1954) and all used, with individual minor variations, scales manufactured by the same electronics firm.

Iowa experience

The Iowa State Highway Commission has used the electronic scale for the collection of dynamic axle-load data for planning purposes exclusively (*5*). They now have three scales in operation and are planning to install seven others. This agency uses a single set of amplifying and recording instruments housed in a converted school bus (*6*). Connection to the load cells of an individual scale is made through a weatherproof junction box mounted on the road shoulder. Iowa plans to use this combination of strategically placed dynamic scales and a mobile recording unit as an eventual replacement for their loadometer crews. Official opinion in this State is that the dynamic weights recorded by the scale represent the actual loads applied to the highway and are therefore more useful in planning and design than are the static axle loads. No attempt has been made to use the scales for enforcement purposes.

Comparisons of static and dynamic weights recorded at the Iowa scales showed that 20 percent of the vehicles were "overweighed" by more than 5 percent, while only 5 percent were "underweighed" by more than 5 percent. Dynamic weights recorded for 50 percent of the vehicles fell within 2 percent of the corresponding static weights.

Though few maintenance problems have been encountered in Iowa, the need has been expressed for an improved method of recording the data. A continuous output in the form of a punched card or printed tape would reduce the many hours of tedious work now required to extract weight data from the oscillograph tapes.

Minnesota experience

The Minnesota Department of Highways has been using the electronic scale for all three of the purposes listed above (*7*). An installation near St. Paul uses one scale as a detector to cull out overloaded trucks, and another located 1,500 feet down the highway as a static scale for check weighing the suspected violators. The detector scale is set so that the passage of a truck whose indicated weight is above the legal limit causes a sign to flash on, directing the vehicle to go to the static scale. The violation is also indicated to the scale operator by a warning light. Fines collected from the operators of overloaded vehicles detected by this setup amounted to nearly $15,500 during a 3-month period. This represents over half of the cost of the entire installation.

A second set of electronic scales, one dynamic and one static, placed in a major highway west of Minneapolis, is used to collect wheel-load data for planning and research. Minnesota highway engineers feel that the collection of this type of information is of such importance as to more than justify the cost of the scales.

Minnesota has found that crossing speeds above 55 miles per hour resulted in inaccurate weight recordings. As this was due to the effect of momentum on the sweep of the recording stylus, the speed limit was reduced to 40 miles per hour in the vicinity of the detector. Traffic was channelized to assure that all of the wheels of a given truck would pass over the detector platform.

Maintenance problems in Minnesota have been chiefly caused by the effects of extreme cold on the recording equipment. This has been lessened by insulating the instrument housings, and by using an instrument van similar to Iowa's (*8*).

Oregon experience

Oregon's experience with weighing moving vehicles began in September 1954, when two electronic scales were placed in a highway near Oregon City (*9*). These scales were used to collect wheel-load data for a long-range pavement performance test.

Calibration was accomplished by using a truck-trailer combination of known static axle weights, and a large trucking concern agreed to run their scheduled vehicles over the scales. This provided a sizable sample of known axle loads, in various combinations of size and spacing. Statistical analysis of the data collected during this period enabled the calculation of "correction factors" for converting the recorded dynamic weights to equivalent static weights. The correction factors averaged about 1,000 pounds for the particular traffic and speed range existing at this location. This was considered of sufficient accuracy for research purposes.

Many maintenance problems arose during the first year of operation of the Oregon scales. A report published at the end of that time stressed the difficulty in leveling the platform. The four-point support requires that the tops of the load cells be adjusted to within 0.001 inch of the same elevation, so that each cell will carry an equal share of the dead load. Oregon's trouble occurred because at the initial placement of the platform, no way was provided to make subsequent leveling adjustments. Difficulties were also precipitated by accumulation of moisture in the scale pits, resulting in deterioration of the load cells and of the circuitry, and by failures in the electronic equipment due to variances in the voltage supply.

The original two scales were eventually removed and reinstalled, along with an additional two, in the Baldock Freeway. These four scales were operated 16 hours each month for a period of 20 months. The same troubles originally encountered continued to preclude the successful operation of the scales. The expense of maintaining the equipment under these conditions became prohibitive, and the weighing operations were discontinued in early 1958 (*10*).

Other installations

On the Indiana Toll Road, a slightly different type of scale is now being used at the toll plazas for detecting overloaded vehicles. This scale was manufactured by a firm specializing in toll collection equipment. The steel platform is supported on four moisture-proof load cells, and can be adjusted positionally in three directions by a system of tie bolts. The weight sensing elements of the scale can be preset to a given weight limit, and cause an alarm to be sounded when the limit is exceeded. Also featured is an automatic print-out, recording the vehicle's passage, time, place of entry, and other pertinent data.

Other potentially fruitful studies have been under way in the United States. Among these are the electronic scale installations used for research at the AASHO Road Test and at the Michigan State Highway Department Research Laboratory, and the study of road loading mechanics conducted at the Cornell Aeronautical Laboratory.

The setup at the AASHO Road Test, which was not built specifically for weighing vehicles in motion, consists of three scales. Two of these are used for the static weighing of the tandem axles of the test vehicles. The third scale has served as a calibration instrument for the tire-pressure measuring equipment used in the study of variations in pavement loading. Informal tests conducted by the scale manufacturer, shortly after construction was completed, showed differences of up to 20 percent between the static and dynamic weights of a 30,000-pound axle. Although the physical facilities and instrumentation of the Road Test offered a unique opportunity for full exploration of the problem of dynamic weighing, the primary interest has necessarily been in other areas, and the electronic scale has served only as a useful tool.

While the AASHO Road Test is mainly concerned with the effect of the vehicle upon the road, Michigan engineers are investigating the reverse situation; that is, the stresses and strains induced in a vehicle by varying pavement conditions. The scale used in this work has a 7-foot long steel platform which weighs approximately the same as a section of pavement of the same dimensions. Impact is introduced by placing wood strips of various thicknesses on the platform during the test runs. Dynamic weights recorded by the scale are used to check those obtained through tire-pressure variations on the test vehicle.

The Cornell Aeronautical Laboratory, in its study of road loading mechanics (*11*), has been investigating the cause and effect of the high percentage of variation of dynamic axle weights from their corresponding static weights. These studies have been made by the use of mathematical models representing typical vehicles and road surface conditions. The preliminary findings show that for the same surface roughness, the maximum ratio between dynamic weight and static weight may range from 1.5 for a rear loaded truck with shock absorbers to 7.5 for an empty truck with no shock absorbers. Actual experience with dynamic weighing, however, has not revealed consistent variations of this size, although differences of 20 to 50 percent are common.

Other American Dynamic Scales

Several attempts have been made to weigh moving vehicles by means other than the floating-platform electronic scale. Work of this nature has been the subject of at least two theses for civil engineering master's degrees in the last 4 years.

With the perfection of the resistance wire strain gage, and instruments to detect and record its output, it became feasible to measure the actual stresses produced in a structure both by stationary and by moving loads. One of the first studies of this type was conducted by the University of California on the San Leandro Creek Bridge near Oakland (*12*).

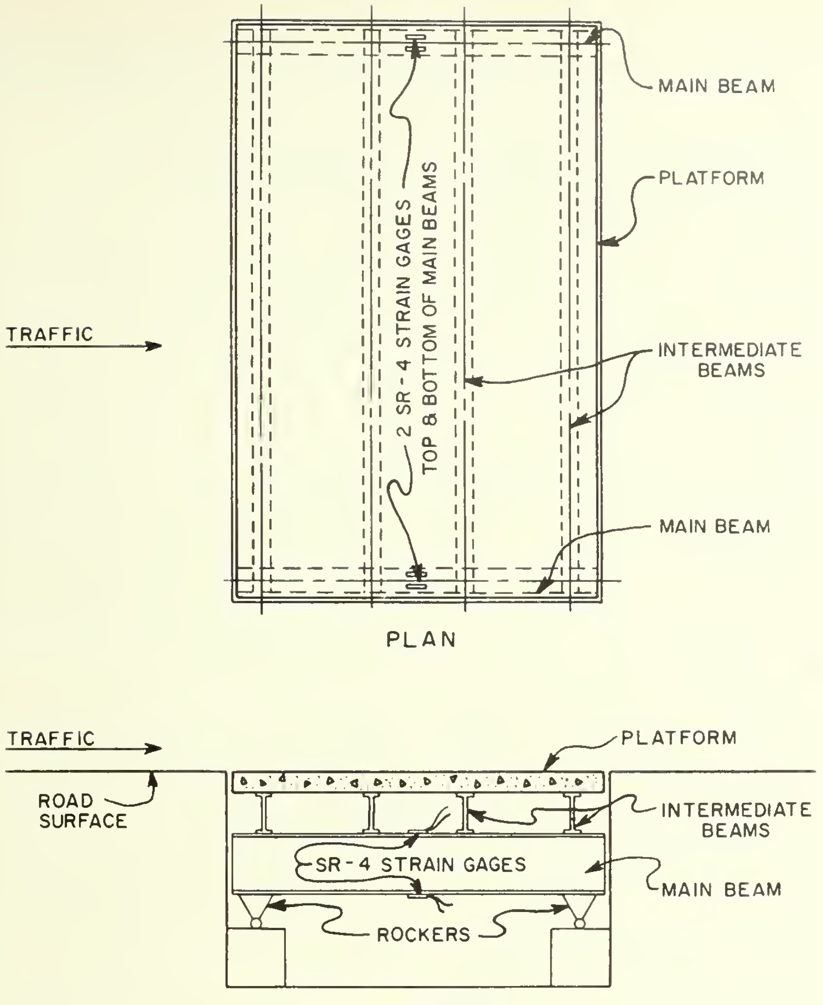


Figure 3.—Experimental beam-type scale.

Part of the study was concerned with the effect of dynamic loads transmitted to the bridge structure by a moving vehicle. This was done very effectively by attaching strain gages to the supporting girders and using a recording galvanometer to present the gage output in a form which could be interpreted.

This same process was used by a University of Kentucky graduate student to weigh moving vehicles (*13*). Strain gages were attached to the steel I beams supporting the platform of an existing lever-type scale, as shown in figure 3. The weight of a vehicle moving over the platform resulted in a momentary strain in the beams due to bending. The gage output, under strain, was recorded by a light-beam galvanometer similar to that used in the San Leandro study. It was reasoned that the deflections of the galvanometer would be proportional to the applied load.

After calibration, test runs were made with a heavy vehicle moving across the platform at speeds varying from 10 to 50 miles per hour. The results showed variations ranging to over 30 percent between dynamic and static weight. This experiment indicated that it was practical to measure dynamic loads through the bending in a simple beam, and opened the way for a more thorough investigation of the possibilities of the use of the strain gage as a weight-measuring device in this manner.

Dynamic weighing has also been the subject of a thesis by a Mississippi State College student (*14*), who designed and built a portable scale consisting of a series of small, flat load cells sandwiched between two 17- by 41-inch metal plates. Each load cell was fabricated from a ½-inch length of 2-inch diameter steel shaft. Load was applied to each cell through a spherical segment cut from a ball bearing, embedded (flat side down) in the top center of the cell. The load was measured by a strain gage attached to a recess cut into the bottom of the cell opposite the bearing point. The scale was embedded in the pavement along the right wheel track of the traffic lane and was tested by running a truck over it at speeds of 10, 20, and 30 miles per hour. The output of the scale's load cells was fed into a high-speed recorder which indicated the results on paper tape. Since the stylus deflection of the recorder did not bear a linear relationship to the applied load, it was not possible to obtain a good check on the accuracy of this experimental scale. Low cost of constructing the scale, portability, and particularly the fact that no pit is required, would seem to justify further study of a device of this type.

European Dynamic Scales

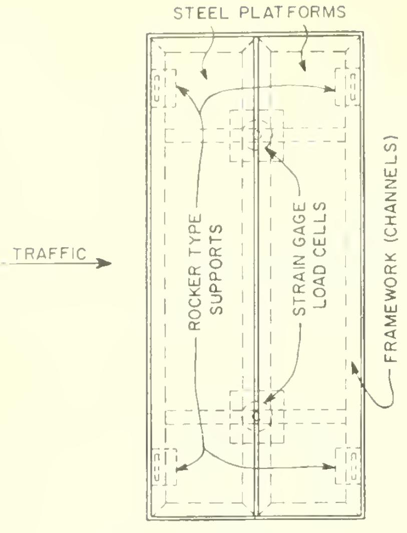
Some of the most promising advances in the field of dynamic weighing have taken place in England and Germany. Here the primary interest has been in the collection of axle-load data for research and planning purposes.

The Road Research Laboratory in England has recently built an electronic scale which will detect a moving axle load and classify it into one of eleven weight groups (*15*). The scale employs a platform and load cells similar to those used in the United States, but the entire unit is built of lightweight materials for maximum portability. Its recording equipment consists of a mirror galvanometer, 11 photoelectric cells, and the same number of mechanical registers. The output of the scale's load cells causes the galvanometer to deflect, moving a beam of light over the bank of photo cells. The photo cells are connected through a transistor circuit to the mechanical registers. The registers automatically increase their count by one as the light beam activates successive photo cells. The upper limit reached by the deflected beam, therefore, represents the weight group into which the axle load falls. The final readings of the registers represent the sum of the number of loads in each weight group, and the total number of loads falling into higher groups. The actual number of loads in each group is obtained by subtraction.

The major advantage of this scale over most of the American models is the possibility of automatic recording over a longer period of time. The results can be read directly without further interpretation. Accuracy is rather limited, and the percent of error ranged above 10 percent for this scale.

What is perhaps the most sophisticated dynamic scale in use today was developed and built in 1958 by a West German research organization (*15*), which was called upon to design a scale that would measure the axle weights of moving vehicles in such a way that: (1) Axles could be weighed separately, even at high speeds; (2) axle loads could be grouped, as with the British scale; (3) results could be read as a total count in each group; and (4) continuous weighing and recording would be possible.

To meet these criteria, a "broken bridge" platform, as illustrated in figure 4, was used instead of the monolithic type common to all known previous scales. The "broken bridge" consists of a steel platform split, at right angles to traffic, into two sections of equal width. The outside edge of each section rests, near the corners, on a hinged support similar to a bridge rocker. The contiguous edges are supported, through interlocking bearing plates, by two strain gage load cells. The platform sections are held in position by heavy springs anchored to the floor of the pit. As a load comes on the platform, the load cell output increases linearly until a maximum is reached when the load is directly over the joint between the two sections. As the load moves off the scale, the cell output decreases linearly to zero. The dynamic weight is therefore



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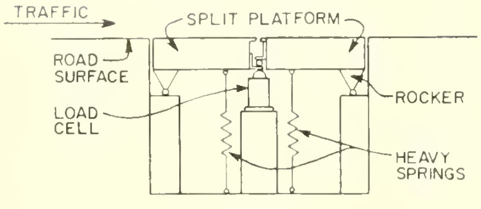

SECTION

Figure 4.—"Broken bridge" scale.

measured by the maximum load cell output recorded at the center of the platform.

The curve representing cell output (weight) is triangular in shape rather than trapezoidal as in the case with the monolithic platform, as shown in figure 5. This permits a more accurate determination of the dynamic weight, as the maximum reading is represented by a sharp peak rather than by some point on an irregularly sloping line. Another advantage of the broken bridge platform is that it does not tend to vibrate as much as the monolithic type under either heavy or light axle loads.

The recording system of the German scale provides for sorting the axle loads into six groups, covering a weight range from 0.3 ton to over 12 tons. A finer division was possible but was not needed in this particular application. The alternating current input to the load cells results in the axle load being represented as an amplitude-modulated signal. This signal is amplified, demodulated, and amplified again. Load curves may be recorded at this point. The signal is then fed into a sorter which places it in the proper load group and records the event through a relay-actuated mechanical counter. The number of loads in each load group may then be read at any time. Other electronic components include optical recorders for each load group, a calibration unit, and a device which automatically corrects zero point drift.

The maximum speed at which individual axle weights can be determined by any electronic scale depends upon the platform length, the response time of the equipment, and the spacing of the axles. For the German scale, with a platform length of 4.6 feet and a counting frequency of 25 counts per second, this top speed is about 78 miles per hour, provided that the axle spacing is greater than 4.6 feet. More closely spaced axles would have to be weighed at much lower speeds if separate weight recordings were desired.

Tests have shown that the dynamic weights obtained from the broken bridge scale are generally within 5 percent of the comparable static weights. Recognizing the importance of reducing the impact factor (dynamic weight/static weight) to as near unity as possible, the German engineers stress the necessity of a smooth approach to the scale platform.

The broken bridge scale described here has been in operation on highway B29 near Grumbach since 1958, and similar installations are planned for major highways near Darmstadt and Dusseldorf.

Conclusions

Based on a study of the experiences of others, discussions with highway and electronic engineers and scale manufacturers, and from personal inspection of various dynamic weighing facilities, the following conclusions seem valid:

1. A properly designed and instrumented electronic scale will measure the actual load applied to the scale platform within certain limits of accuracy. To state this more specifically:

Static loads may be measured as accurately as with conventional lever system scales.

Individual dynamic axle weights may vary from the corresponding static weights by a large percentage.

The total gross weight of a number of axles passing over a section of highway during an extended period of time (and thus their average weights, in weight groups) can be obtained by dynamic weighing, with accuracy sufficient for research and planning purposes. Several agencies have reported average variations between gross dynamic and static weights of 2 percent or less. Commercially available electronic scales, using strain gage load cells, can perform this particular data-gathering function quite well, although the recording methods need to be improved.

2. In the field of overload detection for enforcement purposes, the performance of the electronic scale has been only fair. For this purpose, it would be desirable to maintain the maximum difference between dynamic and static weights at 5 percent or less. A method of damping the vehicle's oscillations, either directly by providing a smooth approach surface, or indirectly through instrumentation, is needed to reduce the variance to this limit. Results obtained in Germany with the broken bridge scale seem to offer some prospect for solution of this problem.

3. Major maintenance and operation problems encountered in previous dynamic scale installations included the damaging effects of moisture on load cells and circuitry; the difficulty of keeping the platform level; movement of the platform upon impact of the vehicle; platform vibration under tandem axles; unsatisfactory recording methods; and the necessity for constant attendance by a skilled operator. Most of these problems have been attacked with varying degrees of success by both the scale manufacturers and the users. Specifications for a model installation should use the findings from this previous work and provide additional solutions where needed.

Future Research Planned

The University of Kentucky research team has planned the study of several dynamic weighing devices. One of these will be a commercially available electronic scale, the testing of which will emphasize the effects of weather and constant traffic upon the per-

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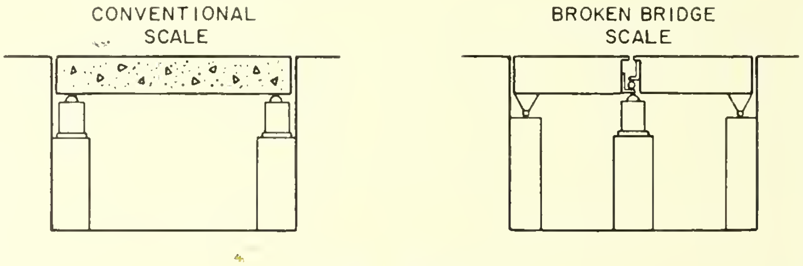

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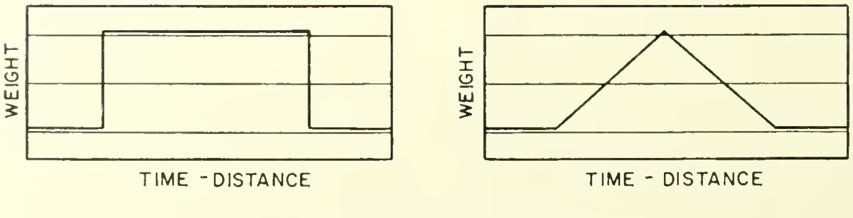

TYPICAL DYNAMIC WEIGHT RECORDINGS

Figure 5.—Comparison of conventional and broken bridge scale dynamic weight recordings.

Concrete Containing Fly Ash as a Replacement for Portland Blast-Furnace Slag Cement

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM E. GRIEB
and DONALD O. WOOLF,
Highway Research Engineers

This article reports a study of the use of fly ash as a replacement for part of the cement in concrete mixes prepared with portland cement and portland blast-furnace slag cement, and tests of a recommended adjusted mix containing fly ash. The results show that fly ash can be used satisfactorily as a replacement for portland or slag cement, but the properties of the concrete are affected by the carbon content of the fly ash. It is indicated that the amount of fly ash used should be correlated with the amount of lime released by the portland cement.

Introduction

A REPORT ON A STUDY of the use of fly ash in concrete was published in PUBLIC ROADS in 1957.[2] That article covered the properties of concrete prepared with each of four fly ashes, as replacements for either one-third or one-sixth of the volume of the portland cement used in the mix. The fly ashes differed widely in carbon content. Two type I portland cements were used, one having a high alkali content and the other a low alkali content.

Following issuance of that report, several questions were raised which resulted in the undertaking of further tests.

One of the questions was whether fly ash could be used as a replacement for part of the cement in concrete mixes prepared with portland blast-furnace slag cement. This cement is defined as an intimately interground mixture of portland cement clinker and granulated blast-furnace slag. The slag content in the final product is limited to a range of from 25 to 65 percent by weight.[3] The finely ground slag is by itself only slightly cementitious. Concern was expressed over the possibility that a further dilution of the cement constituent, by replacing part of this cement with fly ash, might cause the concrete to have unfavorable strength characteristics. Although fly ash has a pozzolanic reaction with lime liberated by portland cement during hydration, there was a question of whether sufficient lime would be available for this reaction.

In the first part of the investigation reported in this article, tests identified as Series I were made on concretes prepared with portland cements and with portland blast-furnace slag cements in which one-third of the cement (by volume) was replaced by fly ash.

The investigation reported in 1957 had indicated that a direct replacement of either one-sixth or one-third of the portland cement by fly ash resulted in reduced strength of the concrete at all ages through 28 days. It was also found that replacement of one-third of the cement by fly ash adversely affected the resistance of concrete to attack by calcium chloride used for ice removal.

In a discussion of that study, a producer of fly ash recommended the use of an adjusted mix containing fly ash, which reportedly would furnish good early strength and resistance to scaling in concrete. In this mix, the cement content was reduced from 6.0 to 5.25 bags per cubic yard, the sand-to-total aggregate ratio was reduced by 2 percent, and an addition of 100 pounds (approximately 1⅓ bags) of fly ash was made. To determine the effect of this mix on the durability and strength of concrete, tests identified in this article as Series II were made.

Conclusions

These two series of tests verify some of the conclusions of the previous report: mainly, that the properties of the concrete are affected by the carbon content of the fly ash used as a replacement for part of the cement. Lower strengths and durability were obtained with concrete containing the high-carbon fly ash than when the low-carbon fly ash was used.

Fly ash can be used as a replacement for a part of the portland blast-furnace slag cement without serious decrease in the strength or durability of the concrete.

Concrete prepared with low-carbon fly ash as a replacement for portland cement, in the amounts recommended by the producers of fly ash, has compressive and flexural strengths at 28 days equal to similar concrete without fly ash.

The recommended amount of fly ash, if low in carbon content, also produces concrete which has resistance to scaling, caused by the use of de-icing agents, equal to that of similar concrete without fly ash.

Tests for strength and durability of concrete indicated that, for optimum effectiveness, the amount of fly ash used should be correlated with the amount of lime released by the portland cement. If sufficient lime is not available to react with all of the fly ash, some of the latter is only an inert filler, and tends to impair the quality of the concrete.

It is suggested that the amount of fly ash used should be about 15 percent by weight of the calcium oxide content of the cement, as determined by chemical analysis. This amount may vary somewhat according to the chemical composition of the aggregates.

Materials

In this investigation, tests for compressive and flexural strength and for durability were made on air-entrained concrete mixes prepared with two fly ashes and four cements. The fly ashes were similar to those used in the previous investigation, but different cements were used.

The two fly ashes used were identified as B and Y, and corresponded to fly ashes B and Y used in the previous investigation. Fly ash B had a low-carbon content (0.6 percent) and fly ash Y had a high-carbon content (11.2 percent). ASTM Specification C 350–60 T limits the loss on ignition of fly ash to 12 percent. (The loss on ignition is due mainly to carbon.) Both fly ashes met this requirement.

The cements used were a type I portland cement and a type IS portland blast-furnace slag cement, from each of two cement plants. At each plant, the same clinker was used in the manufacture of the two types of cement. These cements were identified as cements B–I and B–IS, and E–I and E–IS, and corresponded to the cements of the same identification reported in a previous investigation of portland blast-furnace slag cement in concrete.[4] The two cements from source B were low-alkali cements and those from source E were high in alkali. The chemical composi-

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[1] Presented at the 64th annual meeting of the American Society for Testing Materials, Atlantic City, N.J., June 26–30, 1961.

[2] *Use of Fly Ash in Concrete*, by A. G. Timms and W. E. Grieb; PUBLIC ROADS, vol. 29, No. 6, Feb. 1957, p. 142.

[3] Definition of portland blast-furnace slag cement in ASTM C 205–58 T.

[4] *Tests of Concrete Containing Portland Blast-Furnace Slag Cement*, by W. E. Grieb and George Werner; PUBLIC ROADS, vol. 31, No. 9, Aug. 1961, p. 183.

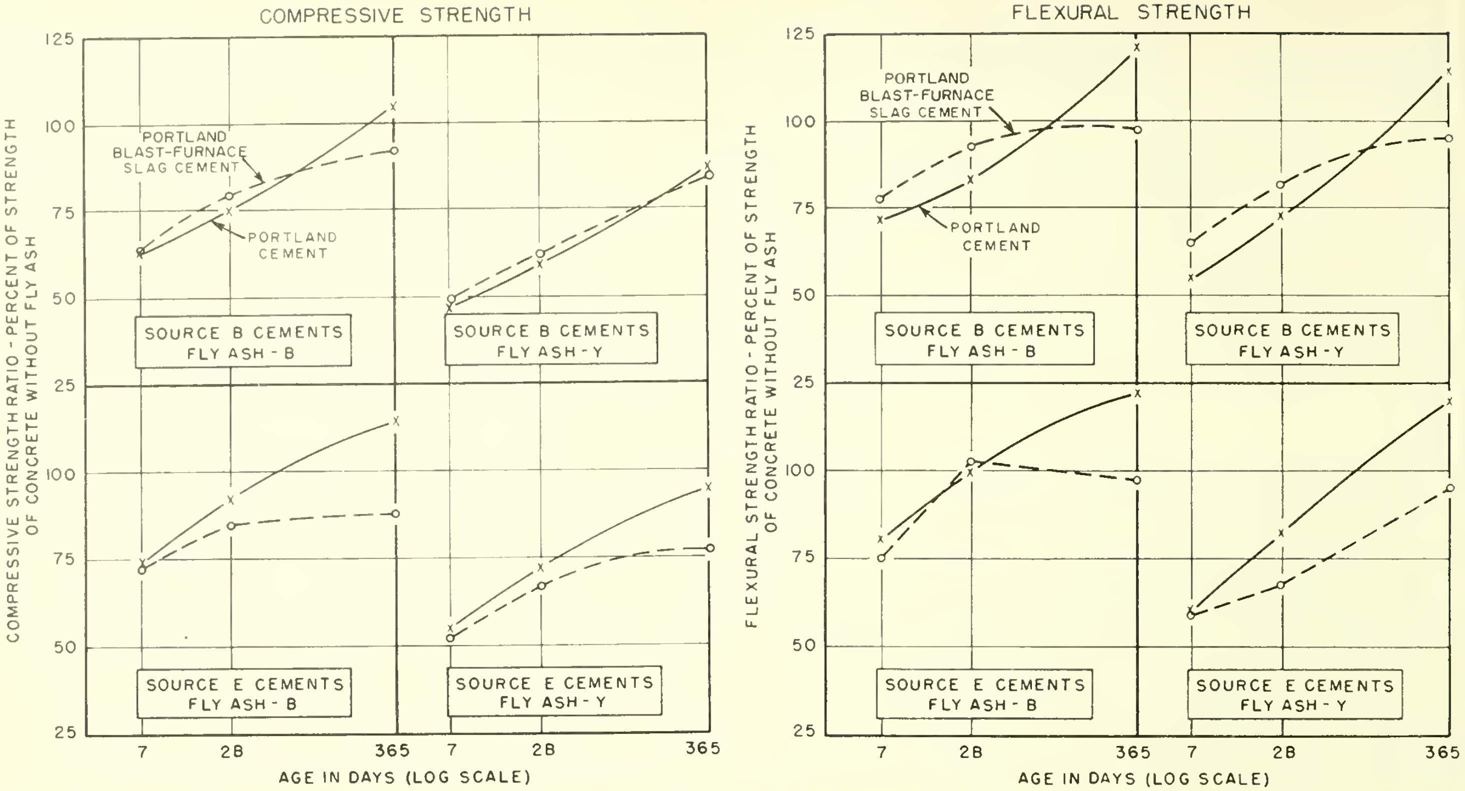

Figure 1.—Effect of replacement of one-third of portland or portland blast-furnace slag cement by fly ash on strength of concrete.

tion and physical properties of these fly ashes and cements are given in table 1.

The aggregates were similar to those used in the previous investigation. They were a siliceous sand and a crushed limestone. The concretes for strength tests were made with 1½-inch maximum size coarse aggregate, and for durability tests the maximum size was 1 inch. A commercially available aqueous solution of neutralized Vinsol resin was used to entrain air.

Mix Proportions

The mix data for all concrete specimens are given in table 2. All tests were made on air-entrained concrete. The control mixes without fly ash were designed for a cement content of 6.0 bags of cement per cubic yard of concrete with a slump of 3 inches and an air content of 5 percent.

In the Series I tests, one-third of the cement was replaced by fly ash; thus in the 6-bag mix, 2 bags of cement were replaced by an equal solid volume of fly ash. Because of the differences in the specific gravities of the materials, 74 pounds of fly ash B or 72 pounds of fly ash Y was equivalent to the solid volume of a 94-pound bag of cement. The weights of the fine and coarse aggregates per cubic yard of concrete were the same for the mixes with and without fly ash.

One-third replacement of fly ash is more than would ordinarily be used in highway construction. However, if this amount could be used without detrimental effects, it would be obvious that lesser amounts could also be used.

For the Series II tests, the control mixes without fly ash were the same as those used in Series I. Where fly ash was used, the mix was adjusted as recommended by the producers of fly ash. Three-quarters of a bag of cement (70.5 pounds) was replaced by 1⅓ bags of fly ash (100 pounds), and to compensate for the greater volume of fly ash as compared with the volume of cement replaced, the sand content was reduced by approximately 45 pounds per cubic yard. The weight of the coarse aggregate per cubic yard of concrete was the same for the mixes with and without fly ash.

Table 2 shows that for both Series I and II, the mixes prepared with fly ash B generally required slightly less water than was required for the corresponding mix without fly ash. Where fly ash Y was used, slightly more water was generally required than for the mix without fly ash. This was true both for the con-

Table 1.—Chemical composition and physical properties of cements and fly ashes

	Portland cements		Portland blast-furnace slag cements		Fly ashe	
	B–I	E–I	B–IS	E–IS	B	Y
Chemical composition (percent):						
Silicon dioxide	20.8	21.3	26.2	26.1	49.2	38.5
Aluminum oxide	5.1	5.6	7.8	6.7	19.9	23.5
Ferric oxide	3.1	3.5	2.3	2.4	16.2	18.8
Calcium oxide	63.6	62.5	53.6	55.7	5.5	3.2
Magnesium oxide	2.8	1.7	3.6	2.8	1.4	1.0
Sulfur trioxide	1.8	2.3	2.3	2.2	2.7	.6
Loss on ignition	1.6	1.3	2.0	2.2	[1] 1.2	[1] 11.6
Sodium oxide	.09	.15	.09	.15	2.00	.60
Potassium oxide	.15	1.02	.20	.91	2.35	1.88
Total equivalent alkalies as Na_2O	.19	.82	.22	.75	3.55	1.84
Insoluble residue	.16	.20	.73	.26	------	------
Chloroform-soluble	.003	.003	.006	.008	------	------
Carbon	------	------	------	------	.6	11.2
Computed compound (percent):						
Tricalcium silicate	57	43	------	------	------	------
Dicalcium silicate	17	28	------	------	------	------
Tricalcium aluminate	8	9	------	------	------	------
Tetracalcium aluminoferrite	9	11	------	------	------	------
Calcium sulfate	3.1	3.9	------	------	------	------
Slag used in manufacture (percent)	------	------	45	35	------	------
Physical properties:						
Apparent specific gravity	3.12	3.14	3.03	3.03	2.52	2.43
Specific surface (Blaine) cm.²/g	3,325	3,555	4,820	3,605	4,305	3,220
Passing No. 325 sieve percent	87.5	96.2	97.6	94.9	95.2	89.4
Autoclave expansion percent	0.06	0.04	0.00	0.00	------	------
Normal consistency percent	23.8	26.5	27.2	26.6	------	------
Time of setting (Gillmore):						
Initial hours	3.7	2.9	3.4	2.8	------	------
Final hours	5.6	4.8	6.2	6.2	------	------
Compressive strength (1:2.75 mortar):						
At 7 days p.s.i	2,625	3,525	2,630	2,360	------	------
At 28 days p.s.i	4,150	4,965	5,190	3,900	------	------
Mortar air content percent	10.4	9.4	7.4	9.2	------	------

[1] Determination made at 600° C.

Table 2.—Concrete mix data

Cement			Fly ash		Mixes for strength tests [1]					Mixes for durability tests [1]				
Identification		Amount	Identification	Amount used	Water used	Slump	Weight of plastic concrete	Air content	A.E.A. used [2]	Water used	Slump	Weight of plastic concrete	Air content	A.E.A. used [2]
Source	Type													
SERIES I														
		Bags/cu. yd.		*Lb./cu. yd.*	*Gal./cu. yd.*	*In.*	*Lb./cu. ft.*	*Pct.*	*Pct.*	*Gal./cu. yd.*	*In.*	*Lb./cu. ft.*	*Pct.*	*Pct.*
B	I	6.0	None	None	31.2	3.1	146.8	4.6	100	34.4	2.4	145.5	4.9	100
		4.0	B	148	31.0	3.3	146.2	4.7	202	31.3	2.0	144.9	5.8	171
		4.0	Y	144	31.5	3.0	145.4	5.2	617	35.8	2.9	144.7	4.2	500
B	IS	6.0	None	None	31.9	2.9	147.2	4.8	100	35.3	2.9	145.5	4.2	100
		4.0	B	148	29.8	2.9	146.8	4.2	104	35.2	2.7	143.5	5.9	129
		4.0	Y	144	31.5	2.7	145.1	4.8	239	34.1	2.6	140.2	6.7	321
E	I	6.0	None	None	31.5	2.9	147.4	5.4	100	34.5	2.6	145.5	5.1	100
		4.0	B	148	31.3	2.9	147.5	4.7	138	32.7	2.5	145.5	5.2	157
		4.0	Y	144	32.1	3.0	145.9	5.1	447	35.9	2.6	144.7	4.9	429
E	IS	6.0	None	None	31.6	2.8	146.4	5.1	100	35.0	2.8	143.5	5.8	100
		4.0	B	148	30.4	3.3	146.6	4.6	169	33.0	3.2	142.7	5.5	143
		4.0	Y	144	31.7	3.0	146.7	4.2	434	36.6	3.2	142.7	4.5	386
SERIES II														
B	I	6.0	None	None	31.2	3.1	146.8	4.6	100	34.4	2.4	145.5	4.9	100
		5.25	B	100	31.3	3.2	147.2	4.5	183	32.8	2.2	146.9	5.0	143
		5.25	Y	100	31.8	3.2	146.1	4.9	488	35.5	2.4	145.1	4.2	357
B	IS	6.0	None	None	--------	--------	--------	--------	--------	35.3	2.9	145.5	4.2	100
		5.25	B	100	--------	--------	--------	--------	--------	34.2	2.0	146.3	4.2	107
		5.25	Y	100	--------	--------	--------	--------	--------	36.2	2.5	145.7	4.2	214
E	I	6.0	None	None	31.5	2.9	147.4	5.4	100	34.5	2.6	145.5	5.1	100
		5.25	B	100	30.6	2.8	146.6	5.1	173	34.8	2.5	145.5	4.4	129
		5.25	Y	100	32.1	3.0	146.7	4.9	334	36.4	3.1	144.5	4.5	286
E	IS	6.0	None	None	--------	--------	--------	--------	--------	35.0	2.8	143.5	5.8	100
		5.25	B	100	--------	--------	--------	--------	--------	35.2	3.1	143.3	5.5	129
		5.25	Y	100	--------	--------	--------	--------	--------	37.1	3.0	144.1	4.4	314

[1] Proportions for mixes without fly ash were 94-170-350.
[2] Relative amount of air-entraining admixture used; amount in concrete without fly ash considered as 100 percent.

cretes prepared with the portland cements and those prepared with the portland blast-furnace slag cements.

The amount of air-entraining admixture required to obtain air contents of 4 to 7 percent was greater for all mixes containing fly ash replacement than for the corresponding mixes without fly ash. Where fly ash B was used, from 4 to 102 percent more air-entraining admixture was needed; and for fly ash Y the increase was from 139 to 517 percent. The relative amounts varied according to the type and source of the cement, the size of the coarse aggregate, and the carbon content of the fly ash. For the mixes without fly ash, more air-entraining agent was required for the concretes prepared with the slag cement than for those prepared with the portland cement.

Mixing, Fabricating, and Curing of Specimens

All mixing was done in an open-pan-type mixer in accordance with usual laboratory procedures. The fly ash was added to the mix with the cement. All tests on the plastic and hardened concretes were made in accordance with the applicable ASTM method. The specimens prepared for strength tests were 6- by 12-inch cylinders and 6- by 6- by 21-inch beams, which were tested at ages of 7, 28, and 365 days. All specimens were stored in moist air until tested.

Freezing and thawing tests were made on 3- by 4- by 16-inch beams in accordance with the ASTM Method C 292 for slow freezing and thawing in water. Freezing and thawing was continued for 250 cycles or until the specimens showed an average loss in N^2 of 40 percent.

Outdoor exposure tests were made on 16- by 24- by 4-inch slabs. A dam was cast around the top perimeter of each specimen and, when freezing and thawing was expected, the top surface was covered with ¼ to ½ inch of water. Each morning after the water had frozen, flake calcium chloride was spread over the ice at a rate of 2.4 pounds per square yard of surface. After the ice had melted, the calcium chloride solution was washed off and fresh water was left on the surface of each slab. The slabs were examined periodically and rated for amount and depth of scaling of the exposed surface. They were subjected to a total of 40 applications of calcium chloride.

Strength Tests

The results of the compressive and flexural strength tests on concrete with and without fly ash replacement are given in table 3. This table also gives the compressive strength ratio of the concrete prepared with fly ash, expressed as a percentage of the strength of the corresponding concrete without fly ash. This was referred to in the previous report as relative strength.

Average values for the compressive strength ratios of the concrete for Series I at various ages are shown in the left half of figure 1. A direct comparison is shown between the concretes prepared with fly ash and the portland cement and the slag cement from the same source. The upper diagrams show the compressive strength ratios of the concretes prepared with the two types of cement from source B and containing the two fly ashes as replacements for one-third of the cement. These diagrams show that at 7 and 28 days the strength ratios of the slag cement concretes were slightly higher than the strength ratios of the corresponding portland cement concretes, regardless of whether fly ash B or fly ash Y was used as a replacement. At an age of 1 year, the strength ratios of the portland cement concretes were higher than the strength ratios of the slag cement concretes for both fly ashes B and Y. Values given in table 3 show that at 28 days and at 1 year, the actual compressive strengths of the concretes prepared with the slag cement from source B, with and without fly ash, were higher than the actual strengths of the corresponding concretes prepared with portland cement.

The lower diagrams in the left half of figure 1 show the compressive strength ratios of the concretes prepared with the two types of cement from source E, with the two fly ashes as replacements for one-third of the cement. At all ages the strength ratios of the concretes prepared with the portland cement were greater than the strength ratios of the slag cement concretes. At 7 and 28 days there was little difference between the strength ratios of the concrete prepared with the two types of cement, but at 1 year the difference between them was pronounced. Table 3 shows that the actual compressive strengths at all ages were lower for the concrete prepared with the slag cement from source E than for the corresponding portland cement concrete.

Table 3.—Results of strength tests [1]

Cement			Fly ash		Flexural strength			Compressive strength			Strength ratio [2]					
Identification		Amount	Identification	Amount	7 days	28 days	1 year	7 days	28 days	1 year	Flexural strength			Compressive strength		
Source	Type										7 days	28 days	1 year	7 days	28 days	1 year
SERIES I																
		Bag/cu. yd.		*Lb./cu. yd.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
B	I	6.0	None	-- ---	585	755	800	3,580	5,340	6,900	100	100	100	100	100	100
		4.0	B	148	420	630	980	2,240	4,050	7,280	72	83	122	63	76	106
		4.0	Y	144	325	550	920	1,730	3,190	6,040	56	73	115	48	60	88
B	IS	6.0	None	------	525	750	935	3,360	6,180	8,250	100	100	100	100	100	100
		4.0	B	148	410	695	920	2,160	4,950	7,590	78	93	98	64	80	92
		4.0	Y	144	345	615	900	1,660	3,850	6,990	66	82	96	49	62	85
E	I	6.0	None	----	685	780	790	4,340	5,410	6,650	100	100	100	100	100	100
		4.0	B	148	555	780	975	3,210	4,980	7,550	81	100	123	74	92	114
		4.0	Y	144	420	640	950	2,380	3,950	6,290	61	82	120	55	73	95
E	IS	6.0	None	--- -	495	720	935	2,680	4,460	6,450	100	100	100	100	100	100
		4.0	B	148	370	740	920	1,930	3,750	5,670	75	103	98	72	84	88
		4.0	Y	144	290	490	895	1,390	2,980	4,950	59	68	96	52	67	77
SERIES II																
B	I	6.0	None	------	585	755	800	3,580	5,340	6,900	100	100	100	100	100	100
		5.25	B	100	515	750	985	3,350	5,280	8,550	88	99	123	94	99	124
		5.25	Y	100	490	705	955	2,890	4,680	7,680	84	93	119	81	88	111
E	I	6.0	None	------	685	780	790	4,340	5,410	6,650	100	100	100	100	100	100
		5.25	B	100	630	780	895	3,890	5,430	7,740	92	100	113	90	100	116
		5.25	Y	100	590	685	775	3,530	4,920	7.190	86	88	98	81	91	108

---

[1] Each value is average of three tests.

[2] Based on strength of concrete without fly ash (relative strength in previous report).

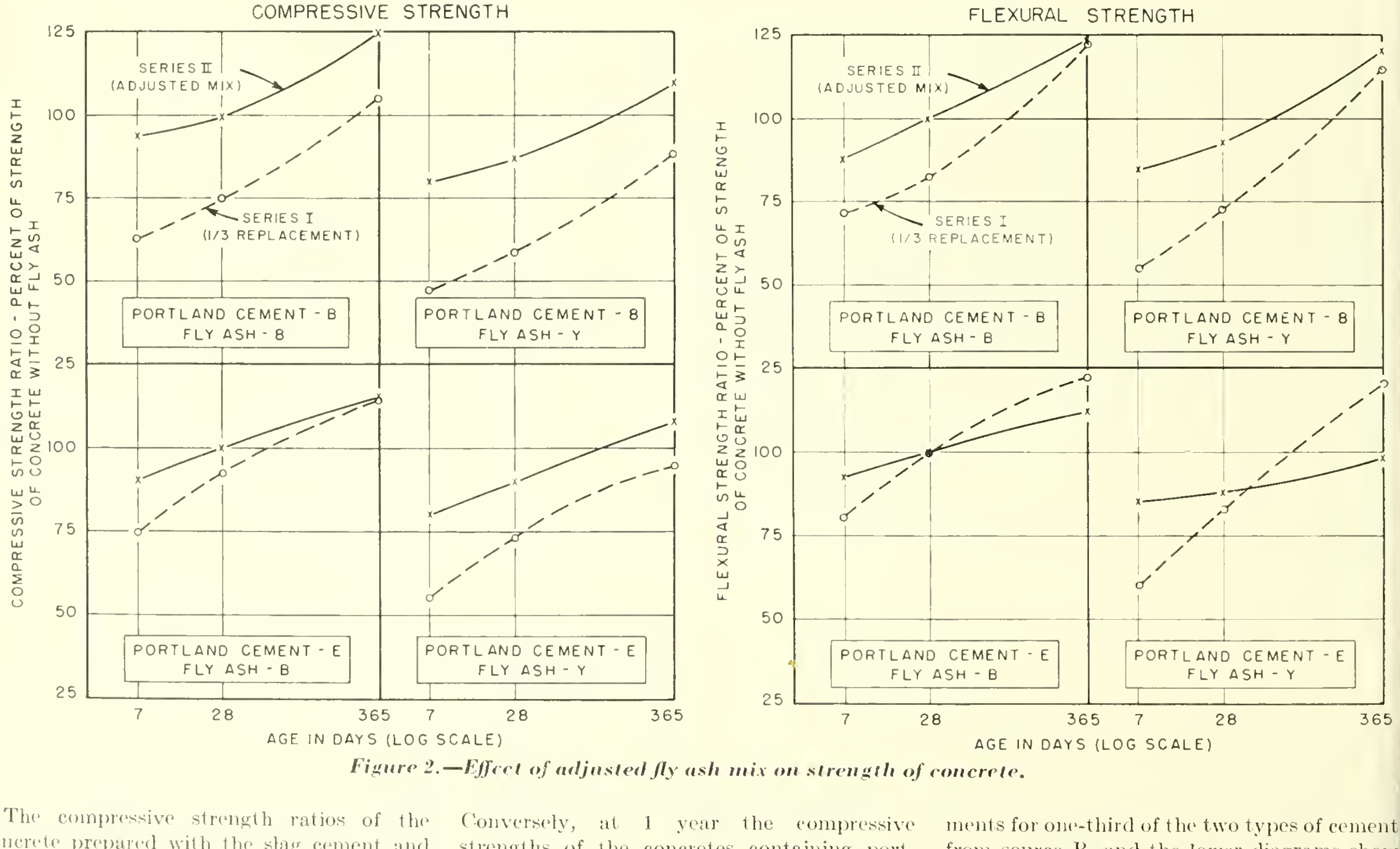

Figure 2.—Effect of adjusted fly ash mix on strength of concrete.

The compressive strength ratios of the concrete prepared with the slag cement and containing one-third fly ash replacement were lower than 100 percent. In the four combinations of cement and fly ash, these ratios increased as the age of the concrete increased. It appears unlikely, however, that these strengths of fly ash concrete would eventually equal the strengths of the corresponding concrete without fly ash. Conversely, at 1 year the compressive strengths of the concretes containing portland cement and fly ash B had exceeded the strengths of the corresponding concretes without fly ash.

The flexural strength ratios of the concretes for Series I are shown in the right half of figure 1. The upper diagrams show the flexural strength ratios of the concretes prepared with the two fly ashes as replacements for one-third of the two types of cement from source B, and the lower diagrams show the flexural strength ratios for concretes prepared with cements from source E.

The same trends are shown for flexural strengths as were shown for compressive strengths: That is, the strength ratios of the concretes prepared with the slag cement from source B were higher at 7 and 28 days and lower at 1 year than the strength ratios of

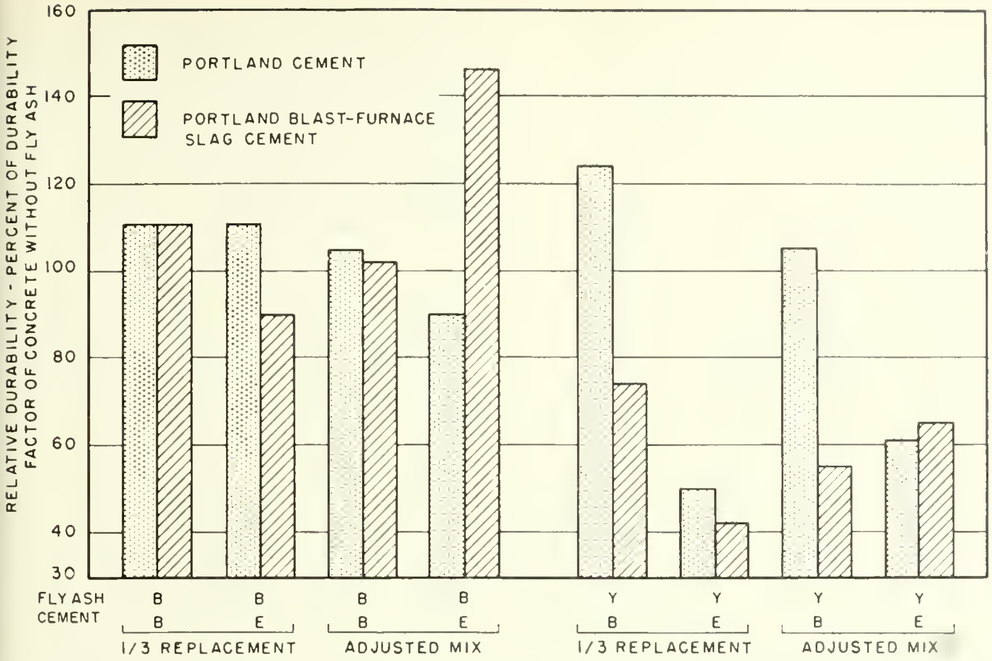

Figure 3.—Effect of replacement of portland or portland blast-furnace slag cement by fly ash on durability of concrete.

corresponding concretes prepared with the portland cement. The strength ratios of the concretes prepared with the slag cement from source E were lower at all ages, except the 28-day test of concrete containing fly ash B. Table 3 shows that at 1 year the actual flexural strengths of the slag cement concretes with one-third fly ash were higher than the actual strengths of the concretes prepared with portland cement without fly ash.

Strength tests using the adjusted mix recommended by the producer of fly ash were made only with the type I portland cement. The results of these tests and the strength ratios of this concrete are given in table 3 as Series II and are shown in figure 2. This figure also shows a comparison between the strength ratios of the concretes for Series II and similar concretes for Series I.

In these tests, the effect on the strength of the concrete of both the source of the cement and of the fly ash is quite marked. As shown in the left half of figure 2, compressive strength ratios were higher at all ages for the adjusted mix of 5.25 bags of cement per cubic yard than for the mix with a cement content of 4.0 bags.

As shown in the right half of figure 2, flexural strength ratios at 7 and 28 days were equal or higher for the adjusted mix, in all cases. At 1 year the adjusted mix for the concretes prepared with cement B with both fly ashes had higher strengths, but when cement E was used the strengths were lower.

The results of these tests support the claim of the fly ash producer that the adjusted mix will give good concrete strengths at early ages. The average compressive strength ratio of the adjusted mixes prepared with fly ash B at 28 days was 100 percent, and for fly ash Y it was approximately 90 percent. Likewise, the flexural strength ratios averaged 100 percent for fly ash B and 90 percent for fly ash Y.

Laboratory Freezing and Thawing Tests

Freezing and thawing tests were made in accordance with ASTM Method C 292 for resistance of concrete specimens to slow freezing and thawing in water. Tests were made using all four cements and both percentage replacements of the two fly ashes. The results of these tests are given in table 4. This table gives the loss in N^2 at 50, 100, 150, 200, and 250 cycles, and the durability factor, calculated on the basis of 250 cycles or a maximum loss in N^2 of 40 percent. In addition, the relative durability of the concrete is given. This is expressed as the ratio of the

Table 4.—Results of laboratory freezing and thawing tests [1]

Cement			Fly ash		Loss in N^2 at cycles shown [2]					Durability factor	Relative durability [3]
Source	Type	Amount	Identification	Amount	50	100	150	200	250		
SERIES I											
		Bag/cu. yd.		*Lb./cu. yd.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*		*Pct.*
B	I	6.0	None	----	1	11	38	(159)	----	38	100
		4.0	B	148	0	6	31	(174)	----	42	111
		4.0	Y	144	1	14	28	(195)	----	47	124
B	IS	6.0	None	----	0	0	28	(195)	----	47	100
		4.0	B	148	0	0	7	32	(215)	52	111
		4.0	Y	144	5	18	(145)	----	----	35	74
E	I	6.0	None	----	0	0	3	11	20	80	100
		4.0	B	148	0	0	0	5	11	89	111
		4.0	Y	144	0	11	33	(166)	----	40	50
E	IS	6.0	None	----	0	0	9	40	(200)	48	100
		4.0	B	148	0	0	21	(178)	----	43	90
		4.0	Y	141	0	(85)	----	----	----	20	42
SERIES II											
B	I	6.0	None	----	1	11	38	(159)	----	38	100
		5.25	B	100	0	6	35	(166)	----	40	105
		5.25	Y	100	0	18	35	(166)	----	40	105
B	IS	6.0	None	----	0	0	28	(195)	----	47	100
		5.25	B	100	0	0	16	10	(200)	48	102
		5.25	Y	100	0	32	(108)	----	----	26	55
E	I	6.0	None	----	0	0	3	11	20	80	100
		5.25	B	100	0	0	1	23	28	72	90
		5.25	Y	100	0	1	12	38	(205)	49	61
E	IS	6.0	None	----	0	0	9	40	(200)	48	100
		5.25	B	100	0	0	5	17	30	70	146
		5.25	Y	100	0	22	(130)	----	----	31	65

[1] Each value is average of three tests.
[2] Figures in parentheses indicate number of cycles at which a loss of 40 percent was reached.
[3] Based on durability of concrete without fly ash.

Table 5.—Results of outdoor scaling test

Cement			Fly ash		Air	Rating [1] after—	
Source	Type	Amount	Identification	Amount		15 cycles	40 cycles
		Bags/cu. yd.		*Lb./cu. yd.*	*Pct.*		
B	I	6.0	None	None	5.3	2½	4½
		5.25	B	100	4.5	3	4½
		5.25	Y	100	4.5	4½	7
E	I	6.0	None	None	4.8	2	4
		5.25	B	100	6.2	2	3
		5.25	Y	100	5.3	5	7½

[1] Rating after freezing and thawing with calcium chloride for indicated number of cycles. A rating of zero represents no scaling; a rating of 10 represents deep scaling over entire surface.

durability factor of the concrete with fly ash replacement to that of the corresponding concrete without fly ash.

The relative durability factors are shown in figure 3. When fly ash B was used as a replacement, the relative durability in all cases was 90 percent or more and in six of eight cases it was over 100 percent. With this fly ash and a cement content of 4.0 or 5.25 bags, more durable concrete was usually obtained than with a 6.0-bag, air-entraining concrete not containing fly ash. When fly ash Y was used, durable concrete was obtained only with portland cement from source B. With this high-carbon fly ash, the concretes prepared with slag cement from source B and both cements from source E showed poor durability.

When all mixes containing fly ash were considered, it was found that those prepared with portland cement had a somewhat greater relative durability (95 percent average) than those prepared with slag cement (86 percent). Comparisons between the fly ash concretes of Series I (4.0 bags of cement) and those of Series II (5.25 bags of cement) showed both to have very nearly the same relative durability. However, the slag cements of Series I had a lower durability factor (79 percent) than those of Series II (92 percent).

These tests indicate that satisfactory durability was obtained for concrete when fly ash B (low-carbon fly ash) was used. This applied to all sources and types of cement used. The tests also indicated that poor durability was usually obtained for concrete when fly ash Y (high-carbon fly ash) was used. Only the concrete prepared with the portland cement from source B gave satisfactory durability with this fly ash.

Outdoor Scaling Tests

Table 5 shows the results of the periodic examinations of the slabs which were exposed to cycles of outdoor freezing and subsequent thawing with calcium chloride. Tests were made on concretes prepared with the portland cements only, and when fly ash was used as a replacement of cement, the amount was that recommended by the producer. The specimens were rated according to the amount and depth of scaling. A rating of zero indicated no scaling and a rating of 10 indicated deep scaling over the entire surface of the specimen.

These tests showed that at 40 cycles the concretes which contained fly ash B had resistance to scaling equal to or slightly better than similar concrete without fly ash. Where fly ash Y was used, however, the resistance was poorer than that of the concrete without fly ash.

Dynamic Weighing of Vehicles

(*Continued from p. 204*)

formance of the scale and the effect of preloading on platform vibrations. Also planned is the design, installation, and testing of at least two types of experimental scales—probably the previously described "broken bridge," and a triangular concrete platform supported by two load cells and a hinge. Continued research is also planned in the development of a beam-supported electronic scale, which utilizes the strain produced by bending as the weight sensor; this will probably use two rectangular aluminum beams as the supporting elements. Corollary to all of this work is further research into the problem of a dependable waterproofing system for the SR–4 strain gage.

All of the test installations will be made in the exit ramp to the north loadometer site on Interstate Route 64, about 50 miles west of Lexington, Ky. Two concrete pits of the proper size, one to receive the commercial scale and the other for the experimental scales, will be built in the right-hand lane of the 24-foot wide ramp pavement. Provisions have been made for a smooth, level approach to the test area. The testing program will include axle-load data collection, overload detection, and diversion of overload vehicles for static weighing.

The Michigan State Highway Department is undertaking a cooperative project with the Bureau of Public Roads, to investigate the overall problem of collecting data on vehicles moving in the traffic stream. These data would include weight, axle spacing, speed, overall length, and other items. It is also proposed to develop a means of transmitting this information to a central location, where it will be automatically analyzed and processed into usable form.

The Michigan and Kentucky projects are intended to complement each other. This joint effort and the efforts of others concerned with the subject should result in solutions to the problems of dynamic weighing, thus providing highway planners with a source of reliable design data and enabling the States to better enforce their load limit regulations.

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Changes in Asphalt Viscosities During Thin-Film Oven and Microfilm Durability Tests

BY THE DIVISION OF PHYSICAL RESEARCH
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This article presents the results of a special study to compare the changes in viscosity and other properties of asphalt occurring during two accelerated aging tests. Data presented include results of the thin-film oven test and the microfilm durability test on a selected group of 11 asphalts of 85–100 penetration grade. These asphalts were refined by a variety of methods from crude petroleum obtained from various sources. Also included is some background information on the development of the thin-film and microfilm tests.

Viscosity measurements for the 85–100 asphalts and their test residues made at several temperatures showed significant differences in temperature susceptibility and degree of complex flow. Two types of behavior were noted: (a) *The hardening in the microfilm test was greater than the hardening in the thin-film test for the majority of the asphalts. This difference was not considered significant for values of aging index of approximately 2 or lower but the difference increased rapidly as the hardening increased.* (b) *The hardening occurring during the two tests was essentially equal at all levels of resistance for the other asphalts.*

This study also illustrated the value of the sliding plate microviscometer for obtaining detailed rheological information on asphalt in research investigations but revealed limitations of the instrument that are a deterrent to its use for control purposes. The differences in asphalt behavior and the changes in relative rheological characteristics at different temperatures or values of stress emphasize the need for specification control of asphalt consistency at more than one temperature.

Further research is required to establish firmly the significance of some of the trends observed in this study. The Bureau of Public Roads has initiated further research with the same group of asphalts in an attempt to relate their properties to the behavior of the laboratory mixtures.

Introduction

AN ACCELERATED LABORATORY weathering test that would properly measure the potential durability of the asphaltic binder in a bituminous pavement has long been the goal of asphalt technologists. At present, two tests are being used that are believed to approach this goal: the thin-film oven test developed in 1940 by the Bureau of Public Roads, and the microfilm durability test developed in 1955 by the Shell Development Company.

In the development of the thin-film test, major emphasis was placed on the temperature conditions existing at the mixing plant; in the development of the microfilm test, an attempt was made to approach the conditions existing in the pavement during service. Obviously, neither test duplicates field or construction conditions exactly; consequently, there has been considerable discussion among asphalt technologists as to the relative significance of the results obtained by each test. The Bureau of Public Roads conducted a study in an attempt to pinpoint the significant differences in degree and type of hardening occurring in both types of tests. This article sets forth some of the findings from tests made during this study.

Development of the sliding plate microviscometer provided a means not only for comparing the results of the two tests on the same basis but also made it possible to obtain data on the more complex rheological properties of the original asphalts and residues. However, limitations of the microviscometer encountered during this study indicate that, while useful for research purposes, adoption of this instrument for control tests appears to be undesirable at this time.

Summary of Findings

This study was not conclusive in all its aspects and further research will be required to establish firmly the significance of some of the trends noted. Observations of principal interest in the study are summarized in the following paragraphs.

Information obtained from several experimental projects indicated a general correlation between the results of the thin-film and microfilm tests for most of the asphalts tested. However, detailed studies of a group of asphalts derived from crude petroleum obtained from various sources and of different methods of refining revealed two types of behavior.

For most of the asphalts tested, the hardening in the microfilm test generally was greater than that occurring in the thin-film test; however, the differences shown by these two tests varied according to the level of hardening. For values of aging index of approximately 2 or lower, the difference was not considered significant; but, as the hardening increased, the hardening in the microfilm test became increasingly greater than that obtained in the thin-film test.

On the other hand, some asphalts showed approximately equal hardening in the two tests at all levels of aging index. However, studies of the viscosity-temperature relationships indicated that the rheological properties of the two residues may differ significantly. The significance of these differences with respect to the behavior of the asphalt as a road binder has not been determined and additional studies are being conducted.

Data obtained with the microviscometer indicated large differences in the degree of complex flow for various asphalts and that the flow characteristics of a given asphalt may change rapidly with temperature changes. Although the relation of these changes to the behavior of pavement mixtures made with asphalt has not been established as yet, it is believed that some coefficient of complex flow will provide a very useful parameter of behavior. Such a coefficient is represented by the slope of the line obtained by plotting the logarithm of the shearing stress against the logarithm of the rate of shear.

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The high degree of complexity of asphalt flow in the range of temperatures most often encountered in the pavement, indicates that the control of consistency at only one temperature is inadequate for specification purposes. Tests at two or more temperatures appear to be desirable.

The change shown in degree of complex flow with temperature changes, as well as differences in viscosity-temperature susceptibility noted, indicated that the substitution of absolute viscosity at a single temperature for control by penetration values may be of more limited advantage than first believed. To obtain more information on this subject, further studies should be conducted over a wider range of temperatures than that employed in this study. For such studies, instruments other than the sliding plate microviscometer will be required.

Description of Tests

The term thin-film test first was applied to the ⅛-inch film test used to denote a comparison with the standard volatilization test. Admittedly, this is not a thin film in the sense of the asphaltic films in a pavement mixture. However, because alternative suggestions for a title are unwieldy, thin-film oven test generally is used to designate the ⅛-inch test. The test developed by the Shell Development Company is referred to either as the microfilm durability test or the aging index test. For convenience and brevity the terms thin-film test and microfilm test are used in this article.

Thin-film test.—The thin-film test is based on the changes that occur in an asphalt when a 50-milliliter sample in a ⅛-inch film is heated in an oven for 5 hours at a temperature of 325° F.

The test has been adopted as an ASTM Tentative Standard (D1764–60T) under the title, *Method of Test for Determining the Effect of Heat and Air on Asphaltic Materials When Exposed in Thin Films.* The American Association of State Highway Officials has also adopted the test and designated it, *Thin-Film Oven Test* (AASHO Method T179). The ASTM and AASHO methods are identical with the exception of the tests to be made on the residues. The ASTM method includes only the penetration of the residue as a designated test, while the AASHO method includes both penetration and ductility of the residue as designated tests. The determination of the weight loss is optional in the ASTM method, but mandatory in the AASHO method.

Microfilm test.—The microfilm durability test, sometimes designated as the aging index test, has not been adopted as an ASTM Standard. Subcommittee B–19 of Committee D–4 has published a proposed method for information, and cooperative studies to determine its precision have been conducted. This test measures the change in absolute viscosity that occurs when a film of asphalt, 5 microns in thickness, is exposed in an oven for 2 hours at 225° F. The result is expressed as the ratio of the final viscosity divided by the original viscosity of the asphalt, both measured at 77° F. The viscosities are determined in each case by the sliding plate microviscometer.

RESUME OF PRIOR RESEARCH

Thin-film tests

Because the thin-film test was first introduced in 1940, some asphalt technologists may not be completely familiar with the original research conducted to establish its usefulness. A brief resume of the major findings of this research are presented here.

The thin-film test was first introduced by Lewis and Welborn of the Bureau of Public Roads (*1*).[2] The first report was based on findings of tests on 50–60 and 85–100 penetration grade asphalts that were representative of asphalt production in the United States at that time. Results were included on tests made for 2, 5, 7, and 10 hours at film thicknesses of ⅛, 1/16, and 1/32 inch. These tests showed that the changes occurring in the ⅛-inch film at 5 hours most nearly reproduced the changes occurring in the Shattuck mixing test, a test that was developed for measuring the hardening of asphalt in a laboratory mixture. Changes other than in penetration also were shown to be generally of the same order of magnitude for the ⅛-inch, 5-hour test and the Shattuck test. Accordingly, this ⅛-inch film thickness and 5-hour exposure time were selected for further study. From this work, it was concluded that the ability of the asphalt to withstand hardening during mixing in commercial operations could be measured by the thin-film test. Further, the report suggested that this property was related to the hardening that occurred during service. The report also included data to show that the standard oven loss test is of little value for predicting the degree of hardening of asphalt that occurs during the mixing process or in service.

A second Public Roads report, made in 1946 by Lewis and Halstead (*2*), presented the results of thin-film tests on asphalts of the 60–70, 100–120, and 120–150 grades obtained from the same sources as the asphalts tested for the first report. The data of the second report generally supported the findings of the earlier report for the 50–60 and 85–100 grades, thereby indicating a general applicability of this test to all grades of paving asphalts.

A third Public Roads report, by Pauls and Welborn, published in 1952 (*3*), compared the hardening of asphalts as measured by the abrasion tests, weathering-strength tests, the Shattuck test, outdoor exposure, and the thin-film oven test. The results of all these tests, in which mixtures were used to evaluate hardening, were shown to be related very closely to the hardening that occurred in the asphalt. Because the thin-film test is a direct measure of hardening and is not complicated by extraction and recovery processes, it was considered to be the most suitable test to use for specification purposes.

The most recent Public Roads studies on asphalts representing production from all parts of the United States were reported in 1959 (*4*) and 1960 (*5*). The data from these reports indicate that the percentage of retained penetration generally tends to be lower for the higher penetration asphalts than for those with low penetration; therefore, for equal severity, different limits for each grade of asphalt should be used in establishing specification requirements for asphalt cement.

It is believed that the combined findings of the foregoing reports, as well as other investigations, support the conclusions that (1) relative resistance to change during heating and mixing is an important property of paving asphalt, (2) a test to measure this resistance is needed in specifications, and (3) the thin-film test fulfills this need.

Microfilm tests

Because the research concerning the microfilm test is more recent than that for the thin-film test, it may be more familiar to those presently engaged in asphalt research. One of the first reports on a microfilm test was presented by Griffin, Miles, and Penther in 1955 (*6*). These authors based their studies on some earlier work by Labout and van Oort (*7*).

Generally, the microfilm test is considered to be a measure of the potential durability of the asphalt in service. The factors of variability in design and construction affect asphalt behavior to a considerable extent. However, results for experimental construction projects, in which variables other than the asphalt have been eliminated as much as possible, indicate a general correlation between behavior of asphalts in microfilm tests and in road behavior.

Relation of Microfilm and Thin-film Test Results

Data are available from several published reports that provide a general comparison of the results of the microfilm and thin-film tests. Results for asphalts used in the Zaca-Wigmore tests in California are contained in two reports. One, the study by Simpson, Griffin, and Miles (*8*), includes the microfilm test results and the other, by Skog (*9*), includes the thin-film test results. Results from both tests on another group of asphalts used in Texas experimental projects are included in a report by Jimenez and Galloway (*10*).

The data taken from these reports are shown in table 1. Several laboratories, including that of the Bureau of Public Roads, cooperated with the Texas Transportation Institute in making tests on duplicate samples of the materials used in the Texas experimental roads reported on by Jimenez and Galloway.

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[2] References cited by italic numbers in parentheses are listed on page 217.

Table 1.—Comparison of thin-film and microfilm test results

Asphalt identification	Original penetration, 100 g. 5 sec. at 77° F.	Thin-film test, percent retained penetration	Micro-film test, aging index
ZACA-WIGMORE PROJECT			
A	225	57	8.5
A-2	218	56	9.6
B-1	255	47	17.1
B-2	223	43	65.4
C	247	54	17.8
C-2	246	51	15.8
D	243	52	15.8
D-2	228	53	7.3
E	227	30	428.0
E-2	239	21	282.0
F	215	40	29.6
G	238	41	33.2
G-2	215	39	59.0
H	230	50	14.3
H-2	212	47	27.4
J	237	58	7.8
TEXAS PROJECT			
1	167	63	4.2
2	172	65	3.1
3	128	58	5.0
4	202	58	5.2
5	202	39	12.2
6	156	49	6.2
7	131	63	4.1
8	123	62	4.8
9	233	30	19.4
10	145	65	3.9
11	163	52	7.0

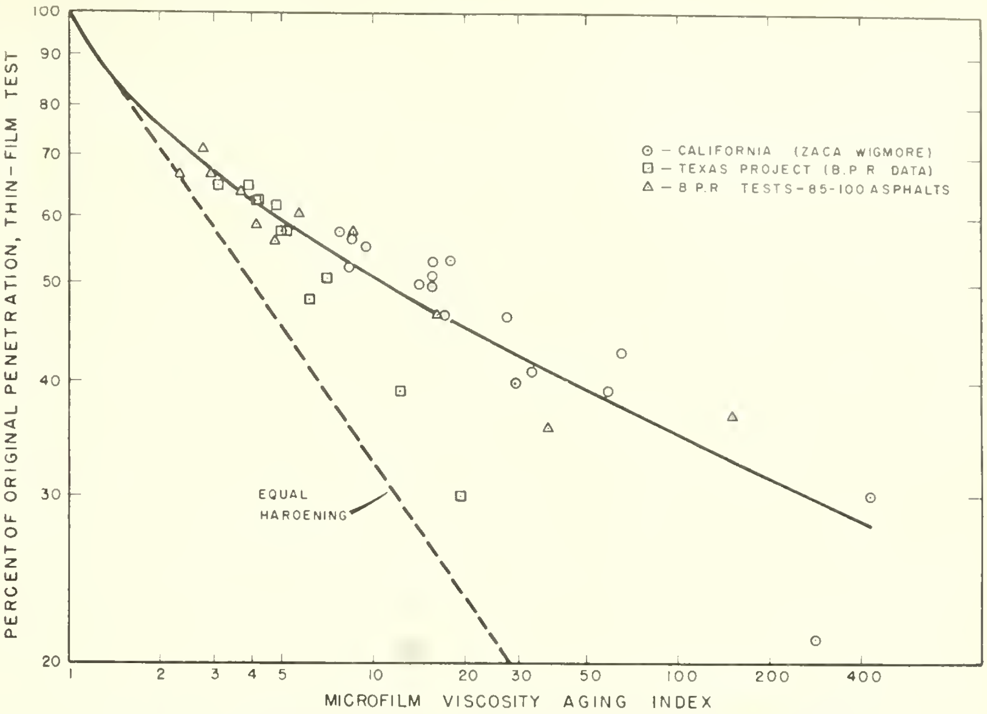

Figure 1.—Relation between results of thin-film and microfilm tests.

The data supplied by Public Roads were selected for presentation in table 1. The thin-film test results selected from the California samples (Zaca-Wigmore project) were the averages of the results obtained by several cooperating laboratories, as reported by Skog.

Figure 1 shows the percentage of retained penetration obtained in the thin-film test plotted against the viscosity aging index obtained in the microfilm test, using a logarithmic scale on both axes. (This figure also presents data for 85–100 penetration grade asphalts selected for the Public Roads special study, which will be discussed later.)

The penetration of the original asphalts from which these data were obtained varied from 85 to 250 yet, despite this relatively wide variation, a general relation exists between the two test results that approaches a straight line. The dashed line in figure 1 shows the curve that would result if the hardening in the asphalt were the same in both tests. This curve was obtained by calculating the equivalent viscosity for a given penetration using an approximate relation established by tests made in the Bureau of Public Roads laboratory. This relation agrees closely with that reported by Saal, Baas, and Heukelom (*11*). With these equivalents, the aging index for any percentage of retained penetration is easily calculated.

The deviation of the experimental curve from the line indicating equal values shows that the hardening of the asphalt occurring in the microfilm test was greater than that occurring in the thin-film test, and that the difference increased with the increase in the susceptibility of the asphalt to hardening. Because of an indication that the data for some asphalts deviated from the general relation more than could be accounted for by experimental error, Public Roads conducted a special study in an attempt to determine the cause of such deviations. This study was conducted on a group of 11 asphalts of 85–100 penetration grade refined by a variety of methods from petroleum obtained from various sources.

Table 2.—Source, method of refining, and test characteristics of 11 selected 85–100 penetration grade asphalts

Sample No.	Source of crude	Method of refining	Test characteristics					Thin-film oven test—⅛-inch film at 325° F., 5 hrs.			
			Penetration 100 g. 5 sec. at 77° F.	Ductility, 5 cm./min. at 77° F.	Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Loss [1]	Tests on residue		
									Softening point	Ductility, 5 cm./min.	Percent of original penetration
				Cm.	*° F.*	*Sec.*		*Pct.*	*° F.*	*Cm.*	
2	Venezuela	Steam and vacuum refining to grade	88	225	118	137	1.018	2.04	131	250+	38
3	Mexico, Panuco	Atmospheric steam distillation to grade	87	248	121	318	1.037	.55	134	138	60
13	Venezuela	(Not given)	89	175	120	255	1.033	.18	132	175	62
38	Midcontinent	Vacuum distillation to 225-275 penetration; blown at 480° F.—to grade.	90	170	120	189	1.000	.00	129	120	62
53	Midcontinent	Vacuum distillation to 80° F.–90° F. softening point; blown to grade—10 hours.	86	151	119	195	.995	+.03	126	150	69
[2] 56	Kansas	Vacuum distillation to grade	86	166	121	180	1.004	+.03	139	13	62
[3] 69	West Texas crudes	Steam-vacuum distillation to grade	87	112	116	120	1.029	+.02	125	178	61
71	Midcontinent-mixed base	Propane de-asphalting and vacuum distillation (Propane asphalt blended with 300+ penetration).	87	200	116	190	1.010	+.08	121	190	70
100	California, Santa Maria	Steam distillation to grade	88	245	120	228	1.031	1.03	136	125	49
[4] 109	Colorado	Vacuum distillation to grade	87	195	118	100	1.019	+.10	129	63	57
119	Colorado	Propane de-asphalting to hard base; fluxed to grade with flux oil.	92	238	116	123	1.031	2.18	129	250+	38

[1] + values indicate gain in weight.
[2] Xylene-naphtha equivalent 4–8.
[3] Xylene-naphtha equivalent 12–16.
[4] Positive spot in 100 percent xylene.

SPECIAL STUDY OF 85-100 PENETRATION GRADE ASPHALTS

Table 3.—Characteristics of 85–100 penetration grade asphalts before and after laboratory film tests

Sample identification	Original asphalt		Thin-film residue				Microfilm residue	
	Penetration at 77° F.	Viscosity[1] at 77° F. in megapoises v_o	Penetration at 77° F.	Percent original penetration	Viscosity[1] at 77° F. in megapoises v_t	Viscosity aging index v_t/v_o	Viscosity[1] at 77° F. in megapoises v_m	Viscosity aging index v_m/v_o
2	86	0.860	31	36	8.21	9.55	32.5	37.8
3	88	1.04	51	58	4.43	4.26	9.00	8.65
13	88	1.16	54	61	3.77	3.25	6.70	5.78
38	89	1.20	57	64	5.60	4.67	4.44	3.70
53	88	1.20	59	67	3.00	2.48	2.80	2.31
56	78	1.67	51	65	4.70	2.81	4.95	2.96
69	88	1.22	49	56	3.45	2.83	5.90	4.84
71	85	0.879	60	71	2.23	2.53	2.45	2.79
100	90	1.21	42	47	6.32	5.22	19.5	16.1
109	83	1.35	49	59	5.22	3.87	5.50	4.07
119	93	0.78	34	37	5.74	7.36	118	151.0

[1] Calculated at a shear rate of 0.05 sec.$^{-1}$

The materials for the special study were selected from asphalts previously included in the general studies of properties of highway asphalts for which reports have been issued in recent years (*4*, *5*). The same identification numbers are used here to permit convenient reference to the earlier work. The sources of the crude petroleums and methods of refining the asphalts are given in table 2, which also shows some of the more significant test characteristics previously reported (*4*). These asphalts were chosen so as to represent widely different types of crudes and methods of refining, degree of volatility, and asphalt composition.

The special tests on these asphalts for this study included the thin-film oven test and the microfilm durability test. The absolute viscosities of the original asphalts and the residues from each of the film tests were determined by use of the sliding plate microviscometer. Determinations to establish the viscosity-temperature characteristics were made at three different temperatures for all materials and at five temperatures for some asphalts of special interest. The results of these tests are given in table 3 and the first five columns of table 4.

Test results for penetration of the original asphalts and the thin-film residues shown in table 2 and table 3 are not always in agreement because tests reported in table 3 were made subsequent to those shown in table 2. However, the agreement is considered excellent in that retained penetration differed by no more than 2 percentage points except for asphalt samples 69 and 56. For sample 69, the penetration of the residue was 4 points lower and the original penetration was higher than previously reported so that the result for retained penetration differed by 5 percentage points. For sample 56, the penetration of the thin-film residues agreed to within 2 percentage points but the original penetrations differed by 8, the last results being the lower. Because both these changes were in the same direction, the percentage of retained penetration differed by only 3 percent.

Viscosity Aging Indexes

Table 3 summarizes the viscosity and penetration data obtained from the microfilm and thin-film tests at 77° F. A viscosity aging index, which is the viscosity of the aged residue divided by the original viscosity, is shown for each test. Unless specifically stated

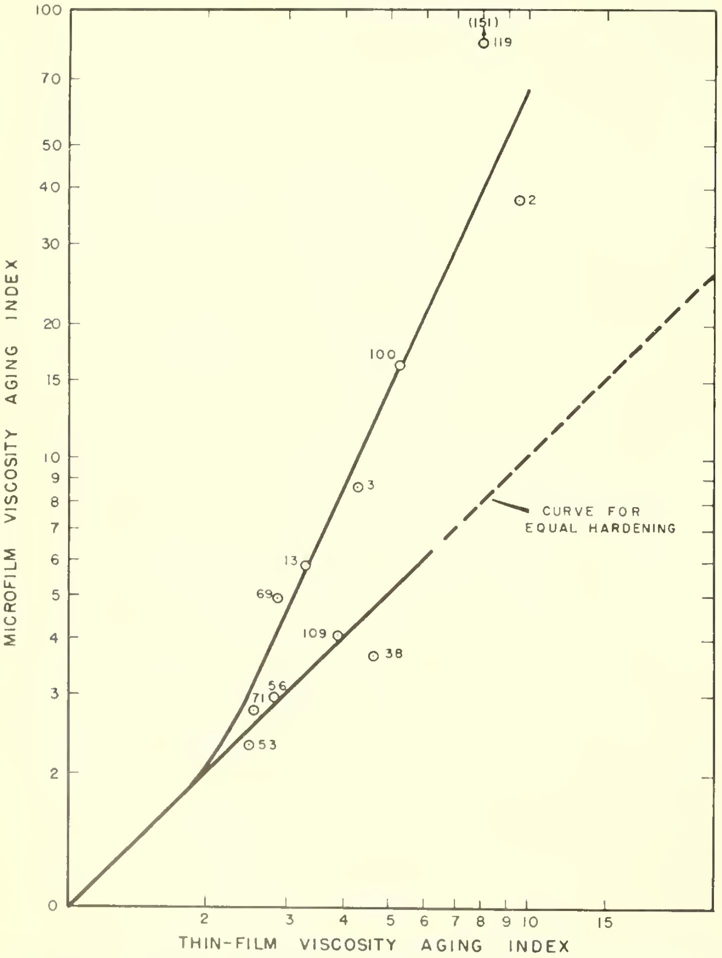

Figure 2.—Relation between viscosity aging indexes for microfilm and thin-film tests.

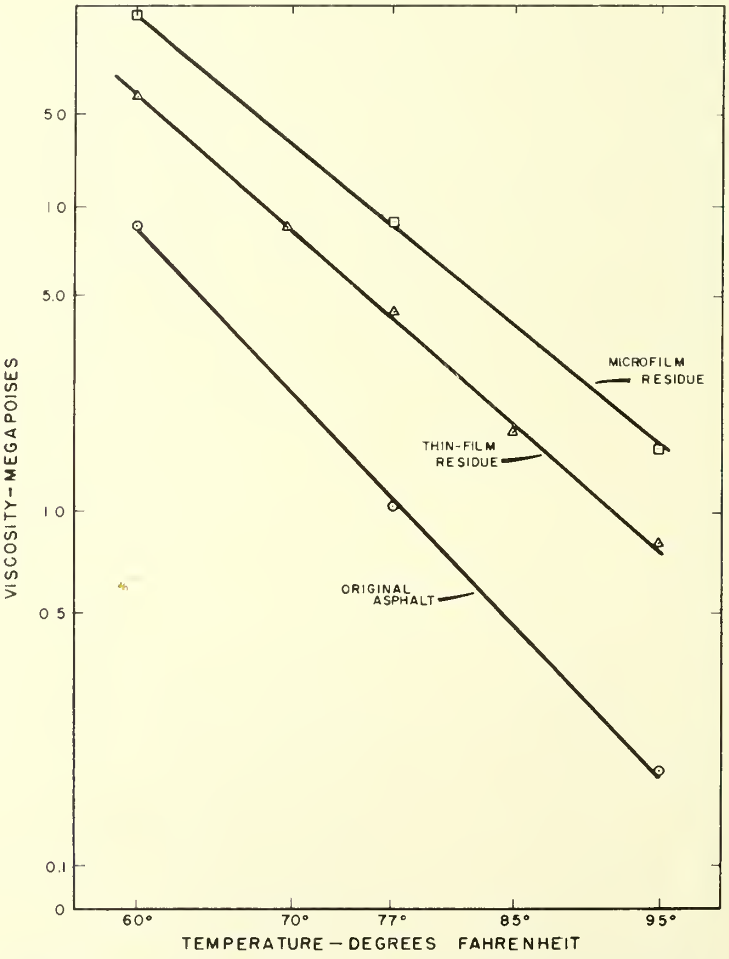

Figure 3.—Viscosity-temperature relations for asphalt sample 3.

otherwise, all viscosities recorded in this table and elsewhere in this article were based on calculations at a shear rate of 0.05 sec.$^{-1}$ Figure 2 shows the viscosity aging index obtained from the microfilm test plotted against that obtained from the thin-film test.

The data presented in figure 2 indicate that for values of aging indexes around 2 there was no significant difference in the results of the two tests. As the resistance to hardening decreased, the increase in viscosity index for the microfilm test increased much more rapidly than that for the thin-film test. However, results for two asphalts very definitely did not follow this trend; essentially the same hardening occurred for asphalt sample 109 in the two tests, but less hardening occurred in the microfilm than in the thin-film test for asphalt sample 38. Check determinations were made for the asphalts showing a different trend; the same results were obtained each time. The results on asphalt samples 53, 56, and 71 showed approximately equal hardening behavior during both tests. However, no large differences would be expected in these three asphalts because of low values of aging index. A consideration of these results, together with data to be presented later, indicates that there were at least two types of behavior of the tested asphaltic materials. In one type of behavior, the difference in hardening of asphalt between the aging indexes obtained by the microfilm and thin-film tests increased exponentially as the index increased; for the other, the hardening indexes were approximately equal.

Viscosity-Temperature Relationship

Viscosities were determined over a range of temperatures to provide information on the temperature susceptibility for the asphalts and the test residues. The data obtained are given in the first five columns of table 4. The range of temperatures used (60° F. to 95° F.) generally represents the limits of the microviscometer for this group of asphalts.

To show the viscosity-temperature relationship, the viscosities were plotted against temperature by using a basis similar to that used for the standard ASTM viscosity-temperature charts. In figures 3–7, the double logarithm (log of the log) of viscosity in poises is plotted against the logarithm of the absolute temperature in degrees Rankine (°F. + 459.7). (For convenience, temperatures are shown in °F on the figures.) The ASTM charts are based on the double logarithm of the viscosity in centistokes. However, for the range of values obtained for these asphalts and because the densities of the asphalts closely approximated 1 gram per cubic centimeter, either scale should give an approximately straight-line relationship for true viscous liquids.

The data points for 7 of the 11 asphalts tested fell in straight lines within small experimental errors. In four cases, which are discussed later, there was some curvature that apparently was not related to experimental error. The viscosity-temperature susceptibility index (m) shown in the seventh column of table 4 is the slope of these plotted lines, except for the four noted. In these cases, the calculations are based on an assumption that the portion of the curve between 70° F. and 85° F. is straight.

Figure 3 shows the viscosity-temperature relations of the original asphalt and the residues from the microfilm and thin-film tests for asphalt sample 3. The curves shown are typical for those asphalts (as mentioned previously) that showed higher aging indexes for the microfilm residues than for the thin-film residues. The temperature susceptibilities of both residues were slightly less than for the original asphalt and, in most cases, the microfilm residue showed slightly less susceptibility than the thin-film residue; but these differences generally were not considered significant. The curves for asphalt 71 in figure 4 indicate that this material generally followed the behavior pattern as shown for sample 3, although the degree of hardening was approximately the same in both the microfilm and the thin-film tests.

Those asphalts showing approximately equal hardening in the thin-film and microfilm tests at all levels are illustrated in figures 5–7 for asphalt samples 38, 56, and 109, respectively. Although the deviation was small for some of these asphalts, the data points do not fall in a straight line. Also, the curves for the microfilm and thin-film residues generally cross. Viscosities determined at 60° F. for the microfilm test residues were higher than those for the thin-film test residues; but the reverse was true at 95° F. At 77° F., the temperature used for the viscosity aging indexes, hardening values were either higher or lower depending on the asphalt.

Because of the curvature of some of the plotted data, it was necessary to base the viscosity-temperature susceptibility index given for these asphalts in table 4 on the portions of the curves between 70° F. and 85° F. Thus, the indexes are not valid over a wide range of temperature, but the values shown are indicative of the trends. The change in slope of the residues compared to that of the original asphalt was generally greater for this group of materials than for asphalts of the other behavior group. This was particularly true for samples 56 and 109.

Complex flow characteristics

It generally is recognized that most asphalts at atmospheric temperature do not have the characteristics of a true liquid (Newtonian flow) but exhibit complex flow; that is, the ratio of the shearing stress to the rate of shear is not a constant. Accordingly, further study of the rheological properties of the asphalts and the residues was made by considering the proposals of Schweyer and Bransford (*12*). These authors, following earlier suggestions by Romberg and Traxler (*13*), recommended that viscosity determinations

Table 4.—Complex flow coefficients and asphalt viscosities at various temperatures and shear rates

Sample identification [1]	Viscosity (0.05 sec.$^{-1}$) in megapoises at—					Viscosity temperature susceptibility index [2]	Viscosity, 1,000 ergs per sec. per cm.^{3} in megapoises at—			Coefficient of complex flow at— c		
	60° F.	70° F.	77° F.	85° F.	95° F.	m	60° F.	77° F.	95° F.	60° F.	77° F.	95° F.
2 *o*	11.1	--------	0.860	--------	0.148	4.8	10.5	0.881	0.122	1.00	1.00	0.65
t	113	--------	8.21	--------	.960	4.7	137	10.7	1.07	.95	.84	.85
m	--------	--------	32.5	--------	2.80	--------	--------	40.4	3.41	--------	.91	.79
3 *o*	8.70	--------	1.04	--------	.181	4.3	9.80	1.10	.200	.92	.92	1.20
t	26.0	8.60	4.43	1.78	.802	3.5	41.4	5.07	.853	.80	.89	.87
m	53.7	--------	9.00	--------	1.55	3.3	127	11.3	1.69	.76	.82	.89
13 *o*	10.6	2.51	1.16	.468	.226	4.1	13.3	1.26	.200	.90	.94	.95
t	24.8	--------	3.77	--------	.573	3.8	36.8	4.08	.578	.86	.95	.96
m	43.7	--------	6.70	--------	1.02	3.6	77.6	7.95	1.03	.88	.91	.95
38 *o*	10.1	2.62	1.20	.470	.132	4.5	12.1	1.28	.122	.81	.86	.88
t	20.7	11.0	5.60	2.87	.940	[3] 3.2	38.8	6.62	.978	.70	.90	.95
m	22.7	9.80	4.44	1.77	.513	[3] 4.1	52.2	5.26	.665	.67	.89	.89
53 *o*	9.80	3.30	1.20	.482	.155	[3] 4.9	11.3	1.22	.116	.91	1.05	.96
t	16.6	6.40	3.00	1.24	.389	[3] 3.9	26.3	3.13	.327	.78	.96	.98
m	16.4	7.20	2.80	1.08	.372	[3] 4.3	35.5	3.00	.295	.70	.91	.92
56 *o*	8.02	3.27	1.67	.675	.206	[3] 4.2	13.2	3.83	.204	.71	.90	.98
t	16.2	7.42	4.70	3.04	1.32	[3] 2.5	48.1	17.4	1.46	.54	.78	.84
m	18.4	7.40	4.95	2.88	1.11	[3] 2.6	60.5	15.4	1.24	.53	.63	.83
69 *o*	20.0	--------	1.22	--------	.150	5.3	20.9	1.25	.158	.99	1.00	1.02
t	34.7	--------	3.45	--------	.352	4.7	50.1	3.41	.356	.87	1.03	.99
m	60.0	--------	5.90	--------	.577	4.7	117	6.56	.541	.77	.91	1.33
71 *o*	10.5	2.50	.879	.311	.0930	5.4	13.4	.909	.0961	.88	.99	1.00
t	18.5	5.40	2.23	.700	.222	4.9	24.4	2.34	.219	.88	.98	.96
m	26.7	--------	2.45	--------	.300	4.7	35.0	2.59	.293	.88	.97	.95
100 *o*	11.2	--------	1.21	--------	.185	4.5	13.3	1.23	.190	.93	1.00	1.00
t	51.5	--------	6.32	--------	.802	4.0	89.6	7.27	.904	.78	.94	.92
m	87.6	--------	19.5	--------	2.53	3.2	443	24.2	3.44	.60	.86	.86
109 *o*	17.2	3.39	1.35	.520	.200	5.0	35.7	1.42	.200	.68	.93	.93
t	24.9	10.3	5.22	2.77	1.57	[3] 3.0	81.6	7.12	1.65	.57	.78	.89
m	25.2	10.4	5.50	2.24	.585	[3] 3.2	71.0	7.63	.605	.62	.68	.94
119 *o*	10.8	2.17	.780	.230	.072	5.7	10.3	.800	.072	1.03	.97	1.00
t	86.0	20.5	5.74	1.67	.540	5.0	123	5.73	.530	.97	1.00	.99
m	--------	--------	118	34.3	9.20	4.5	--------	--------	11.6	--------	--------	.88

[1] o=original asphalt; t=thin-film residues; m=microfilm residues.
[2] Slope of plotted line: log=log viscosity vs. log absolute temperature.
[3] For portion of curve between 70° F. and 85° F.

be reported on the basis of a constant value of the product of shearing stress multiplied by rate of shear, the unit of which is ergs per sec. per cm.[3] or power output per unit of volume. They also recommended that the degree of complex flow be expressed by a coefficient that is equal to the slope of the line obtained when the shearing stress in dynes per cm.^{2} is plotted against the shear rate (sec.$^{-1}$) on logarithmic coordinates. This is based on the general rheological equation:

$$v = F/S^c$$

where,

v = viscosity in poises.
F = shearing stress, dynes per cm.^{2}
S = rate of shear, sec.$^{-1}$
c = coefficient of complex flow.

Experimentally, the values of viscosity and the coefficient of complex flow were determined by plotting on logarithmic coordinates the shearing stress in dynes per cm.^{2} for each loading of the microviscometer against the shear rate (sec.$^{-1}$) obtained at that loading. The actual slope of this line is the coefficient of complex flow. The values of F and S, at the point where this line intersects the line representing $F \times S = 1{,}000$, were used as the basis for calculating the viscosity ($v = F/S$). The results of these computations are given in the right-hand portion of table 4.

The coefficients of complex flow for the asphalts are of particular interest. A value of 1.00 indicates Newtonian flow. It will be noted that in two cases, the computed value significantly exceeded 1.00: the original values for asphalt sample 3 at 95° F. and the microfilm residue for asphalt sample 69 at 95° F. These computations were based on values obtained with only one specimen and such an unusual flow could have been an accidental result or could indicate some instrumental factor.

General Trends of Test Results

Further study is needed to determine the cause of these unusual results and also the repeatability of the values. However, it is believed that the general trends noted are significant. Generally, the degree of complex flow decreased as the temperature increased (the value of the coefficient increased, approaching unity). However, test results for asphalt sample 2 indicated the reverse trend, which might have been an experimental error. It was also noted that the degree of complex flow was usually greater for the test residues than for the original asphalts. The significant exception noted was the unusual behavior of asphalt sample 2 at 95° F., as previously stated.

The viscosity data presented in table 4 show that large differences in reported viscosity can occur because of the basis used for reporting. For example, the viscosity for the microfilm test residue for asphalt sample 100 (at 60° F.) was 87.6 megapoises when calculated on the basis of a shear rate of 0.05 sec.$^{-1}$ but was 443 megapoises when calculated on the basis of 1,000 ergs per sec. per cm.[3]

In these studies the significance of the various differences in asphalts or the causes of such differences were not determined. Public Roads has initiated further research with the same group of asphalts in an attempt to relate some of these properties to the behavior of the asphalt in a mixture. It is hoped that such studies will establish the significance of the differences reported here.

Usefulness of Microviscometer

A secondary, but important, objective of these investigations was to evaluate the usefulness of the sliding plate microviscometer for studying the changes in properties of asphalt. Suggestions have been made that the results of the thin-film test be evaluated on the basis of viscosity changes. Figure 8 (p. 218) shows the relations between the percentage of original penetration and the thin-film viscosity aging index. If a constant relation existed between penetration and viscosity, this plotted curve would be a straight line through the origin. With the possible exception of sample 38, deviations from the plotted curve generally were within experimental error. Therefore, figure 8 provides a basis for converting thin-film test results from percentage of original penetration to a viscosity aging index.

For example, a retained penetration of 47 percent in the thin-film test, the limit used for the 85–100 penetration asphalts in several specifications, is found, by using figure 8,

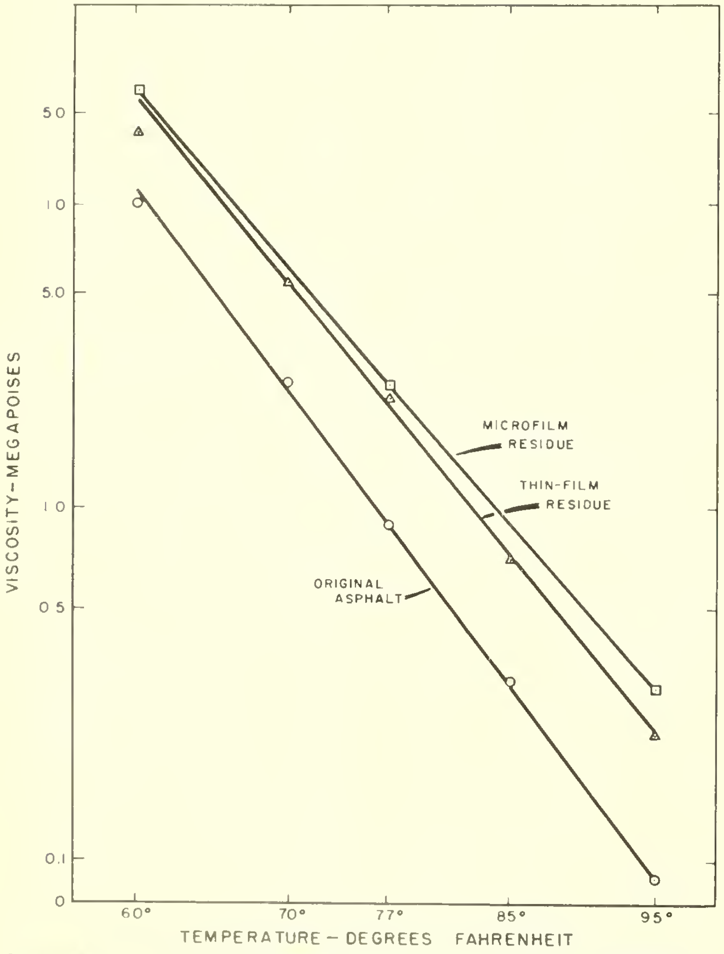

Figure 4.—Viscosity-temperature relations for asphalt sample 71.

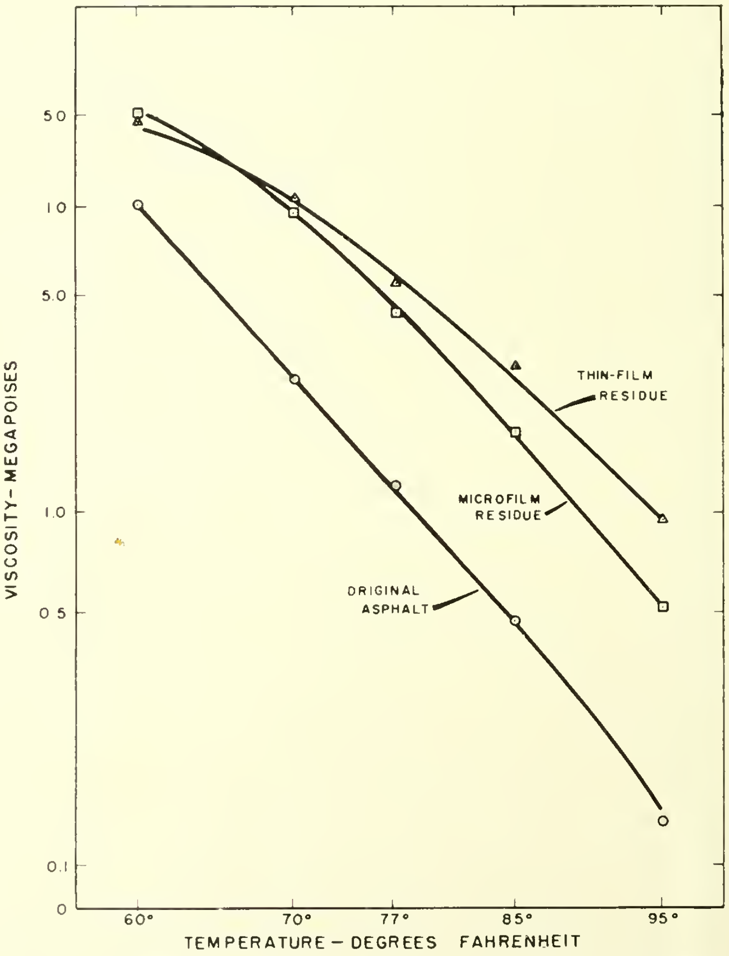

Figure 5.—Viscosity-temperature relations for asphalt sample 38.

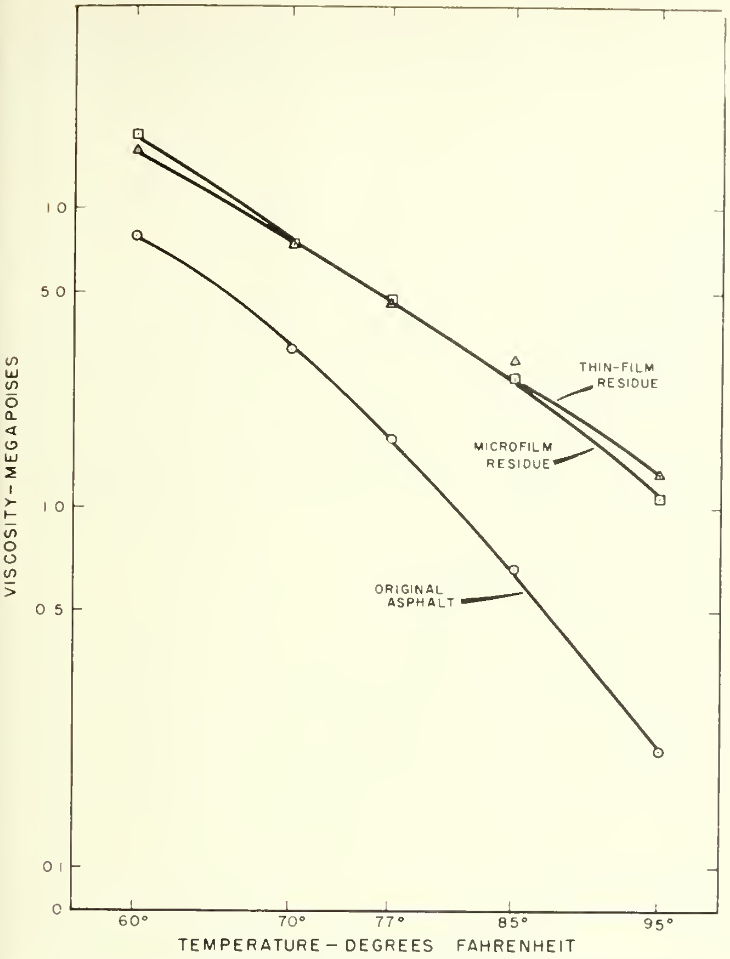

Figure 6.—Viscosity-temperature relations for asphalt sample 56.

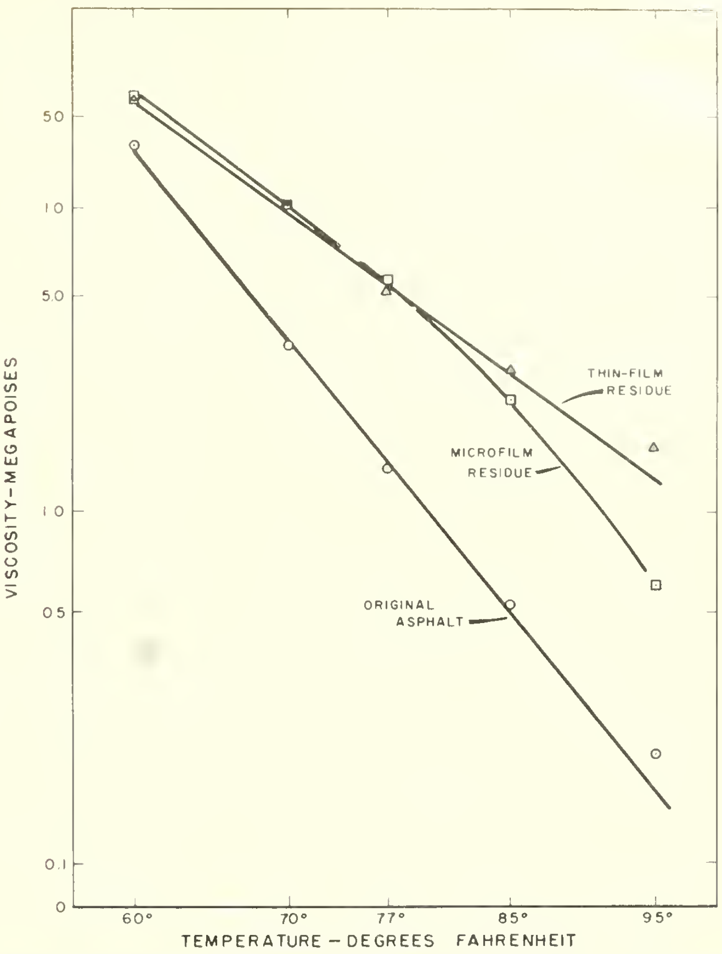

Figure 7.—Viscosity-temperature relations for asphalt sample 109.

to be equivalent to a thin-film viscosity aging index of 4.5. This should not be confused with the equivalent microfilm aging index, which figure 1 shows to be approximately 15.

These tests showed that evaluation of the results of the thin-film test could be made on the basis of absolute viscosities obtained by the sliding plate microviscometer, without materially changing the acceptance or rejection of material. The high degree of technique required for obtaining suitable microviscometry results, plus the fact that only a few laboratories have the equipment, make the recommendation of general adoption of such a procedure undesirable at the present time.

Reports concerning the use of the microviscometer generally tend to emphasize the simplicity of the instrument. Experience with the microviscometer in this investigation indicated that this simplicity has been overemphasized. It was found that an operator needs considerable experience to develop the technique of preparing films so as to obtain repeatable results. The recording instrument requires careful calibration, which must be checked frequently. Determinations for at least four, and preferably five, different loadings must be made at each temperature. Then, computations must be made for each loading; the values must be plotted; and the viscosity must be interpolated for the selected shear rate. For example, each viscosity determination reported in this article required 21 individual computations.

The useful range of the commercial instrument used in these studies was found to be from 0.1 to 100 megapoises for a shear rate of 0.05 sec.$^{-1}$ For values of less than 0.1 megapoise, the instrumental balance became critical and greatly affected the results. For values of about 100 megapoises, the maximum load permitted by the design of the instrument was required. This range of 0.1 to 100 megapoises corresponds approximately to a range of 10 to 300 penetration.

A change in the design of the instrument, such as a horizontal position for the plates or the capacity for greater loading, would permit extension of the range. The upper range might also be extended somewhat if the basis used for reporting viscosity values was changed to viscosities at a constant power output or if lower shear rates were used.

These limitations are pointed out to indicate that adoption of the instrument for control tests appears undesirable at the present time. However, for research purposes it offers many advantages. A considerable amount of information concerning the flow properties of an asphalt can be obtained with this instrument that is not available from empirical tests. It also makes possible more detailed comparisons of changes in asphalt properties induced by laboratory tests and service behavior.

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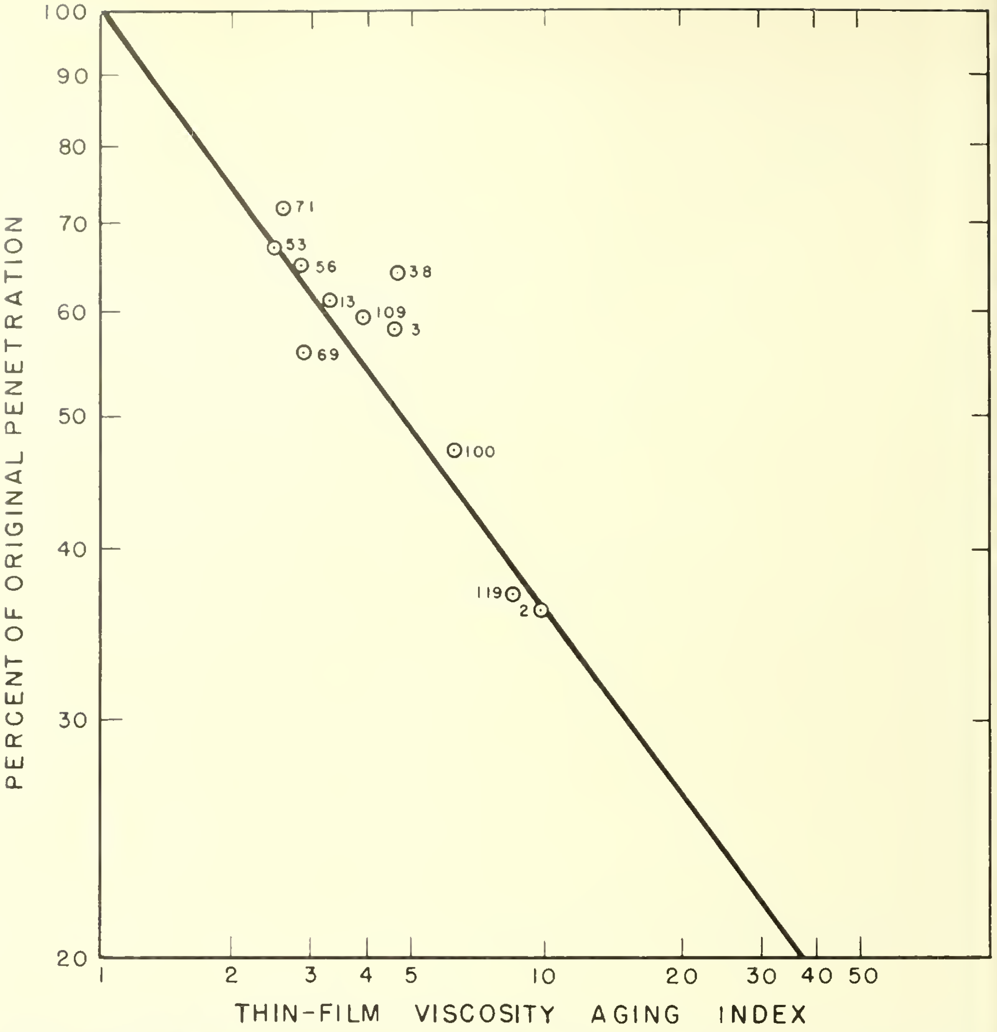

Figure 8.—Relation between percentage of original penetration and viscosity aging index in thin-film test.

Life-Saving Benefits of the Interstate System

by **CHARLES W. PRISK, Special Assistant to the Assistant Commissioner for Research Bureau of Public Roads**

The Interstate System, with its many advanced design features including control of access, will have a key role in highway safety. A reappraisal of its value in this respect indicates that in 1973, the first year after its scheduled completion, more than 5,000 lives will be saved by virtue of the safety features of the Interstate System.

Introduction

IN EACH YEAR OF the past decade, traffic accidents on all roads and streets in the United States have taken 35,000 to 40,000 lives. The total annual number of fatalities has fluctuated within this range during the decade and while it has been increasing somewhat during the past few years, this increase has by no means matched the concurrent growth in the number of vehicles registered annually and the miles that they travel. A reasonable and widely accepted measure of highway safety is the number of fatalities per 100 million vehicle-miles of travel. With this measure showing a fairly steady decline from 7.5 in 1951 to 5.4 in 1959, the record of progress in highway safety provides some encouragement.[1]

There is much greater promise for the future. The 41,000-mile National System of Interstate and Defense Highways, now being built with a target date of 1972 for completion, represents only 1.2 percent of the Nation's total road and street mileage, but it will carry 21 percent of all motor-vehicle travel. The Interstate System, therefore, has a key role in highway safety. Features being built into the system, such as control of access, divided roadways independently designed, and carefully planned interchanges, are expected to have a favorable and lasting impact on the traffic accident problem.

From examination of available facts, it is conservatively estimated that completion of the Interstate System will each year thereafter spare the lives of at least 5,000 persons who otherwise would have died in traffic accidents. This new estimate is 25 percent higher than an earlier estimate of 4,000 made some few years ago.

The accident experience on highways having Interstate System freeway design has already been carefully compared with that on conventional highways carrying major traffic loads in the same area (*1*, *2*).[2] From those data, based on studies of selected facilities in the 1950's, the effect of controlled access and other Interstate System design features on traffic fatality rates is shown in the inset bar chart in figure 1. In urban areas the Interstate type of highway has a fatality rate of 2.0 deaths per 100 million vehicle-miles of travel, as compared with 4.0 for conventional highways; in rural areas the rates were 3.3 and 8.7, respectively.

From analysis of these data, national trends in accidents, and forecasts of traffic, it appears that Interstate freeways are about two-and-one-half times as safe as the highways of earlier design they are replacing. This is an overall relationship. The nature of the benefits varies somewhat between the city and the open countryside. Judged by the number of accidents and deaths per 100 million vehicle-miles of travel, the total accident reduction benefits of freeways are greater in the cities, but their life-saving values are higher in the rural areas.

The Interstate System is scheduled for completion in 1972, and 1973 will be the first full year during which the entire 41,000-mile system will have been in use. In that year, according to estimates from the highway cost allocation study (*3*), travel on the Interstate System will amount to 81.5 billion vehicle-miles on its urban portions and 147.4 billion on its rural segments. In the same year, incidentally, total travel on all roads and streets is estimated at 444.1 billion urban and 651.8 billion rural vehicle-miles.

It is a conservative assumption that fatality rates on the Interstate System will remain essentially constant, although some modest improvement may occur as the motoring public becomes more accustomed to freeway driving and as anticipated refinements in design and operating practices are accomplished. It does seem reasonable to forecast a small reduction of fatality rates in rural areas but the urban rate is already so low that only a slight change could be expected at best. Accordingly, for the purposes of this estimate,

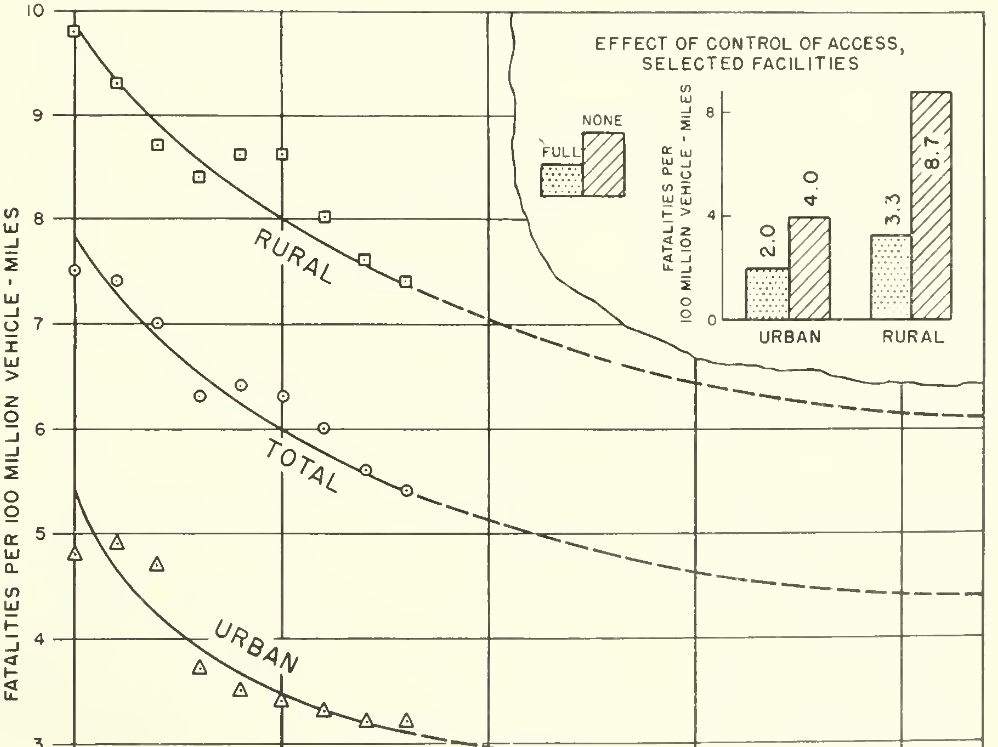

Figure 1.—Highway accident fatality rates in the 1950's, and projection to 1973.

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[1] At the time this article was prepared, 1960 data were not yet available.

[2] References cited by italic numbers in parentheses are listed on page 220.

a value of 2.0 fatalities per 100 million vehicle-miles of travel is assumed for the urban portions of the Interstate System in 1973, and 3.0 for the rural portions.

Measurement for Safety

As a yardstick for measuring the safety benefits of the Interstate System, it also was necessary to estimate the probable fatality rates for all roads and streets in 1973. Figure 1 shows, for urban, rural, and total travel, the fatality rates for all roads and streets for each of the years 1951–59, as reported by the National Safety Council (*4*). During this period the rates declined with reasonable uniformity from 4.8 to 3.2 fatalities per 100 million vehicle-miles of urban travel and from 9.8 to 7.4 for rural travel. (These are akin to the 4.0 and 8.7 indicated for selected facilities without control of access, shown in the bar chart.)

With this background trend, it is anticipated that the accident and fatality rates for all roads and streets will decline further during the next decade. However, it is unlikely that the average rates of decline of the 1951–59 period can continue. As the rates drop, it becomes increasingly difficult to reduce them further. The urban fatality rate is already low, and expansion of urban areas—that is, growth of suburban areas—tends to bring the urban death rate closer to that of the rural areas, which is typically higher.

In figure 1, curves fitted by eye to the plotted fatality rate points for 1951–59 have been projected to 1973, with the assumption that by that year the annual change in rates will be small. The 1973 rates, thus projected, are 2.8 for urban travel and 6.1 for rural.

The application of the various estimates made is shown in table 1. Simply put, the difference between the 1973 fatality rates for all roads and streets (conventional highways) and for the Interstate System is the benefit anticipated from the latter; and the product of this benefit and the travel on the Interstate System in 1973 (both in the same terms, 100 million vehicle-miles) represents the number of lives saved by the completion of the Interstate System. For rural and urban travel combined, the total is over 5,200. It is believed that this estimate is reasonably conceived and conservatively based.

Table 1.—Calculation of lives expected to be saved by the Interstate System in 1973

	Urban	Rural
Death rate (100 million veh.-mi.):		
Conventional highways	2.8	6.1
Interstate highways	2.0	3.0
Benefit (difference)	0.8	3.1
Interstate travel (100 million veh.-mi.)	815	1,474
Total lives saved (benefit×travel)	652	4,569
Total lives saved, urban and rural	5,221	

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A Laboratory-Field Study of Hot Asphaltic Concrete Wearing Course Mixtures

Reported [1] by JOSEPH F. GOODE, Highway Research Engineer,
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Results of a laboratory-field study of properties and performance of asphaltic concrete pavements, ranging in age from 3 to 12 years, from six test sections of Maryland road construction projects are described in this article.

The Los Angeles abrasion wear loss for the coarse aggregates used in the test sections ranged from 17 to 39 percent. Compaction from construction and traffic both appeared to have caused minor degradation of the aggregates. The degree of degradation was insignificant in most instances and in no instance was sufficient to affect service behavior of the pavements.

Air voids in pavement following construction compaction ranged from 5.6 to 14.5 percent. Traffic compaction produced appreciable reductions in percentage of air voids. Pavement performance and rate of asphalt hardening were related to the amount of air voids. A high percentage of air voids resulted in early deterioration of the pavement. The results of the study tend to confirm that the 6-percent air void criterion of the Bureau of Public Roads mix design procedure is satisfactory.

Introduction

INFORMATION PRESENTED in this article was based on a 13-year laboratory-field study (1947–59) conducted by the Bureau of Public Roads and the Maryland State Roads Commission pertaining to degradation of aggregates occurring in hot asphaltic concrete wearing course mixtures.

Objectives established for the study were to determine: (1) The effect of the gradation and toughness of the aggregate used in asphaltic mixtures on the aggregates' resistance to degradation during construction and after various intervals of pavement service, (2) whether aggregate degradation affected the rate of pavement densification under traffic, and (3) the effect of air voids in the pavement on changes in the physical characteristics of the contained asphalt. It was anticipated that findings from this study would make it possible to relate the foregoing factors to pavement performance and to establish the validity of the Los Angeles abrasion wear loss test and Public Roads design criteria.

The study was undertaken because only limited published information was available concerning the extent of degradation of dense-graded aggregate in hot asphaltic concrete occurring during construction or from subsequent traffic. The majority of specifications include a requirement based on the Los Angeles abrasion test (AASHO Method T 96 or ASTM Method C 131) for a coarse aggregate of sufficient toughness to resist degradation. The percentage of degradation permitted within this requirement ranges from 30 to 60 percent for maximum wear loss but most specifications limit the maximum wear to 40 percent. At the time this study was undertaken, however, practically no published information was available to justify use of such varying wear loss values in specifications or to establish that the 40 percent commonly used was the most satisfactory maximum value.

It is believed that the anticipated weight and volume of traffic and the gradation of the aggregate also should be considered during preparation of specifications establishing the maximum allowable percentage of loss. Furthermore, it was noted that few specifications were written to permit use of a softer coarse aggregate when aggregate gradations are high in percentage of material passing the No. 10 sieve, which provides a cushion for the coarse aggregate.

When this study was in the planning stage, it was believed that sufficient information could be obtained by using relatively few road test sections representing a wide range in type and toughness of the coarse aggregate used in the hot asphaltic concrete mixture. The test sections were to be selected from actual Maryland road construction projects and the contractor was not to be required to make any deviations from his normal construction operations. It was known that information concerning aggregates would be limited to that obtained for aggregates having a maximum Los Angeles abrasion test wear loss of 45 percent because this is the maximum value permitted by Maryland specifications for the lowest type of asphaltic concrete. A maximum of 40 percent abrasion loss is specified for the coarse aggregates of the higher type of asphaltic concrete.

Test sections

The six test sections selected for this study were on Maryland roads constructed of different wearing course mixtures that conformed to the Maryland specifications, *Bituminous Concrete, Specification "B"*, and *Bituminous Concrete, Bank-Run Gravel Aggregate*. The Los Angeles abrasion wear loss test of the coarse aggregates in these pavements ranged from 17 to 39 percent. The age of the pavements tested ranged from 3 to 12 years at the time of the last sampling in 1959. The locations of the test sections and their designations in this report were: Section 1, on State Route 216 between Laurel and Fort Meade; section 2, on U.S. Route 40 in Frederick; section 3, on State Route 4 near Upper Marlboro; section 4, on State Route 214 near Davidsonville; section 5, on U.S. Route 1 near Laurel; and section 6, on State Route 24 in Belair. The wearing courses were constructed on old pavement structures of both rigid and flexible types.

Conclusions

Specific conclusions concerning this study were not considered warranted because of the many variables encountered and the limited number of test sections. However, the test data did provide sufficient information to justify the statements made in the following paragraphs.

Degradation of aggregates

The amount of degradation that can be tolerated by the aggregate in a hot asphaltic concrete wearing course mixture could not be definitely determined. Only minor degradation of the aggregates occurred in the tested sections as a result of construction compaction or from subsequent traffic. This minor degradation did not affect the service behavior of these pavements.

The degradation of aggregate caused by steel-tired roller compaction during construction was slightly greater in pavement overlying a rigid pavement structure than for that in a similar pavement overlying a flexible structure. In relation to type of underlying structure, the reverse was true for degradation of aggregate caused by traffic.

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[1] Presented at the 64th annual meeting of the American Society for Testing Materials, Atlantic City, N.J., June 1961.

Table 1.—Description of test sections

	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Date constructed	May 1947	June 1947	Aug. 1948	Sept. 1954	May 1955	July 1956
WEARING COURSE MIXTURE:						
Asphalt:						
Penetration grade	85–100	85–100	85–100	85–100	85–100	85–100
Amount, mix basis ... percent	5. 5	5. 5	6. 2	5. 1	5. 1	5. 5
Coarse aggregate	Slag	Crushed stone	Pit-run gravel	Pit-run gravel	Crushed stone	Crushed stone.
Fine aggregate [1]	Stone screenings and sand.	Stone screenings	Sand	Stone screenings and sand.	Stone screenings	Stone screenings and sand.
Aggregate passing sieve [2] —has confusing appearance. Set [2] —						
1-inch ... percent			100	99		100
¾-inch ... do	100	100	99	93	100	99
⅜-inch ... do	83	94	84	75	75	74
No. 4 ... do	71	73	74	64	59	52
No. 10 ... do	59	47	63	51	37	38
No. 40 ... do	32	17	28	21	19	22
No. 200 ... do	6	8	3	4	7	5
PAVEMENT STRUCTURE:						
New wearing course:						
Width ... feet	24	30–36	24	24	46	24
Thickness ... inches	2	2	2	1½	2	1½
New binder course thickness ... do						2
Underlying structure:						
Type	Flexible	Rigid	Rigid	Flexible	Semirigid	Flexible.
Description [3]	Surface treated macadam.	Surface treated P.C.C.	P.C.C	Asphaltic concrete.	Asphaltic concrete on P.C.C.	Asphalt macadam.
COMPACTING EQUIPMENT:						
2-axle tandem roller	One 8-ton and one 10-ton.	One 10-ton	Two 10-ton	One 8-ton	Two 10-ton	One 8-ton.
3-axle tandem roller				One 13-ton	One 14-ton	One 14-ton.
CONSTRUCTION TEMPERATURES (°F.):						
Air	71	89	94	88	78	90
Mix at plant	300	300	250		292	290
Mix at paver hopper	275	275	250	300	280	250
Mix at first roller passage	185	210	140	140	230	250

[1] Sand used was natural sand.
[2] Average result from two samples.
[3] P.C.C. signifies portland cement concrete.

Pavement densities

Based on the molding procedure of AASHO Method T 167 and ASTM Method D 1074, construction specifications requiring pavement density equal to 98 percent of the pavement mixture's density established in laboratory tests does not seem to be unreasonable and a higher percentage might be justifiable. This statement is based on findings that the pavement densities after construction compaction (determined from AASHO Method T 167) ranged from 95 to 100 percent or more of the densities recorded from laboratory tests, and in only two cases was this ratio less than 98 percent. Higher percentages were obtained for pavements constructed over rigid bases than for those constructed over flexible bases.

The results of the study indicated that the ultimate density of a pavement will be approximately 104 percent of the density established by laboratory tests based on the molding procedure used.

Air voids

Rapid hardening of the contained asphalt from pavements included in this study was attributed to a high percentage of air voids in the pavement. To prevent such rapid hardening of the asphalt, and perhaps an early deterioration of the pavement, the asphalt content of the dense-graded surfacing mixture should be set high enough to assure a low percentage of initial air voids in the pavement. Test results from this study indicated that pavement after adequate compaction should have a volume of initial air voids of no greater than 7 percent.

To prevent eventual flushing of asphalt to the surface or an excessive loss in stability of the pavement, the asphalt content of the paving mixture should be low enough to permit retention of a 1 or 2 percent volume of air voids when the pavement has reached its ultimate density. From this study, it is noted that, in a properly compacted pavement, initial air voids of at least 5 percent generally will assure an adequate amount of air voids, 1 or 2 percent, when the pavement reaches its ultimate density.

On the basis of pavement performance and results of laboratory compaction tests, the 6-percent air void criterion of the Bureau of Public Roads mix design procedure, in which the immersion-compression test is used, appears to be satisfactory.

Test Procedures

Each of the six test sections used for this study represented an area of the road covered by one truckload of mixture, an area of about 700 square feet, on a conventional paving construction project. In each instance, the truckload of material selected was one that appeared to represent the contractor's normal construction mixture. All observations presented in this article were based on resules of tests of samples obtained from these loads of mixture and the areas of pavement constructed with them. A description of the composition of the test sections is given in table 1.

All mixtures contained an 85–100 penetration grade of asphalt; otherwise, the mixtures were quite different in composition. Coarse aggregates used included slag, crushed stone, and pit-run gravel. Fine aggregates used included stone screenings, natural sand, and combinations of these two materials. The gradation of the aggregates varied in nominal maximum size from ⅜- to ¾-inch; in percentage passing the No. 10 sieve from

Table 2.—Characteristics of asphalts used in mixtures

	Test section and asphalt identification					
	1	2	3	4	5	6
Producer of asphalt	A	A	B	A	A	B
Original asphalt:						
Specific gravity ... 77° F./77° F	1. 036	1. 017	1. 025	1. 015	1. 019	1. 034
Flash point ... ° F	490	530	531	572	554	490
Softening point ... ° F	120	115	113	118	116	118
Penetration	101	96	97	93	96	91
Ductility ... centimeters	250+	205	200	165	228	250+
Bitumen ... percent	99. 90	99. 83	99. 85	99. 77	99. 93	99. 71
Organic matter, insoluble ... percent	0. 06	0. 09	0. 10	0. 17	0. 07	0. 25
Inorganic matter, insoluble ... percent	0. 04	0. 08	0. 05	0. 06	0. 00	0. 04
After standard oven test:						
Loss ... percent	0. 20	0. 20	0. 10	0. 02	0. 00	0. 02
Penetration	84	78	87	81	84	77
Retained penetration ... percent	83	81	90	87	87	85
After thin-film oven test:						
Loss ... percent	--------	--------	--------	0. 03	0. 00	0. 76
Softening point ... ° F	--------	--------	--------	126	125	133
Penetration	--------	--------	--------	54	60	47
Retained penetration ... percent	--------	--------	--------	58	62	52
Ductility ... centimeters	--------	--------	--------	250+	218	173

Table 3.—Traffic volumes on test sections

	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Traffic lanes	2	[1] 3	2	2	4	2
Date constructed	May 1947	June 1947	Aug. 1948	Sept. 1954	May 1955	July 1956
Average daily traffic for all traffic lanes:						
1947	560	5,760	------	------	------	------
1948	700	6,430	4,290	------	------	------
1949	790	6,610	4,890	------	------	------
1950	830	7,580	5,110	------	------	------
1951	880	8,760	5,260	------	------	------
1952	[2] 940	8,830	5,180	------	------	------
1953	------	9,680	6,010	------	------	------
1954	------	10,870	6,400	4,730	------	------
1955	------	11,500	6,490	2,980	15,560	------
1956	------	12,310	6,220	3,100	16,180	3,880
1957	------	11,420	6,690	2,900	17,550	4,820
1958	------	11,980	6,810	3,020	18,450	5,200
1959	------	12,500	7,420	3,120	15,950	7,480
Average per year [3]	800	9,550	5,900	3,120	16,970	5,140
Estimated average for test section lane [4]	400	5,700	3,000	1,600	3,400	2,600

[1] One-way traffic. One lane is used for parking.
[2] Later traffic not included as study on this section was discontinued in 1952.
[3] From date of construction to date of final sampling.
[4] Section 2 is in right-hand travel lane and section 5 is in left-hand travel lane, and allowance was made for differences in volume of traffic between different traffic lanes. All values are rounded to nearest 100.

37 to 63; and in percentage passing the No. 200 sieve from 3 to 8.

Physical characteristics of the asphalts used in the paving mixtures for each of the six test sections are shown in table 2. For convenience each asphalt is identified by the number of its test section. Thin-film oven tests (AASHO Method T 179) were performed on asphalts 4, 5, and 6. From these data, it will be noted that asphalt 6 had a relatively high loss in weight, 0.76 percent, in the thin-film test, and its retained penetration was considerably below that of the other two asphalts tested. Asphalts 1, 2, and 3 were not subjected to the thin-film oven test and their properties cannot be compared in a similar manner.

The wearing course portion of the six test pavements ranged in thickness from 1½ to 2 inches. Underlying structures consisted of old pavements of portland cement concrete, asphaltic concrete, or asphaltic macadam.

Pavement samples were obtained the day following construction and thereafter at various intervals. These samples were taken in the outer wheel path; they consisted of slabs, about 12 inches square, cut from the pavement with a diamond blade saw. After each sample had been tested for pavement density, it was warmed so that it could be broken down and remixed to obtain a representative portion for use in the centrifuge method of extracting bitumen. The material adjacent to the saw-cut edges was discarded.

Tests also were made on asphalts recovered from samples of loose mixtures and from most samples of pavement, except for the pavement samples taken immediately after construction.

During construction, other samples taken from each project included samples of the asphalt, the aggregates from which the mixture was prepared, and a second sample of loose mixture from the truck. This latter sample was kept hot in an insulated container and delivered to the laboratory for use in preparing laboratory specimens.

The type and weight of rolling equipment used, shown in table 1, was typical of that employed in pavement construction by the State of Maryland.

The temperatures of mixtures as they were prepared and delivered were considered normal. For two test sections, the mixture behind the spreader was not rolled until it had reached a temperature of 140° F.

Traffic intensity over the six sections varied considerably. Table 3 reports recorded traffic data by year for the entire road width and shows the estimated traffic intensity for the particular traffic lane of each test section, the average daily traffic ranging from 400 to 5,700. Test sections 2 and 3 carried 20 and 10 percent

Table 4.—Progressive changes in gradation of aggregate

Section No.	Type of base	Traffic in test lane [1]	Coarse aggregate	Los Angeles abrasion test loss	Age of pavement	Properties of extracted aggregate									Increase in surface area attributed to—	
						Gradation, percentage passing sieve—								Surface area [2]	Construction compaction	Traffic
						¾-inch	⅜-inch	No. 4 a	No. 10 b	No. 20 c	No. 40 d	No. 80 e	No. 200 f			
		A.D.T.		Pct.	Years									Sq. ft./lb.	Sq. ft./lb.	Sq. ft./lb.
1	Flexible	400	Slag	32	(3)	100	83	71	59	49	32	15	6.3	42	------	------
					0	100	86	76	66	54	35	14	7.0	44	2	------
					1.4	100	85	74	63	51	31	14	7.8	44	------	0
					5.4	100	81	75	66	53	33	14	8.2	45	------	1
2	Rigid	5,700	Stone	18	(3)	100	91	73	47	26	17	11	8.0	35	------	------
					0	100	93	74	49	30	22	14	9.8	41	6	------
					1.5	100	94	74	48	30	21	14	10.0	41	------	0
					5.4	100	92	72	48	30	21	14	10.1	42	------	1
					7.9	100	93	74	48	30	20	14	10.2	42	------	1
					9.1	100	92	70	47	28	20	14	9.9	41	------	0
					12.0	100	96	77	49	31	22	15	10.4	43	------	2
3	Rigid	3,000	Pit-run gravel	39	(3)	99	84	74	63	50	28	8	2.9	31	------	------
					0	97	81	73	62	50	29	9	3.5	33	2	------
					4.2	98	86	75	64	52	28	9	4.2	34	------	1
					6.7	97	85	72	62	52	29	9	4.5	34	------	1
					7.9	100	89	76	65	52	30	10	4.9	36	------	3
					10.7	97	87	75	65	52	30	9	4.3	35	------	2
4	Flexible	1,600	Pit-run gravel	39	(3)	93	75	64	51	37	21	9	4.2	29	------	------
					0	90	76	64	51	37	22	9	4.6	30	1	------
					1.8	95	79	68	54	40	24	11	6.5	36	------	6
					4.6	97	80	69	56	42	26	12	5.5	36	------	6
5	Semirigid	3,400	Stone	32	(3)	100	75	59	37	29	19	11	6.8	32	------	------
					0	100	74	58	37	29	20	12	7.3	34	2	------
					1.2	100	83	62	41	31	21	13	7.9	37	------	3
					4.0	100	79	60	40	32	22	14	7.2	36	------	2
6	Flexible	2,600	Stone	17	(3)	99	74	55	38	30	22	11	5.4	31	------	------
					0	100	72	55	40	33	24	12	5.7	33	2	------
					1.4	100	77	58	41	34	26	13	5.5	34	------	1
					2.8	98	78	62	45	36	26	13	6.1	36	------	2

[1] Approximate. See table 3.
[2] Derived from formula: Surface area $=2+0.02a+0.07b+0.11c+0.26d+0.66e+1.66f$. (See symbols underneath sieve sizes.) This formula gives surface areas comparable to those obtained by the California method.
[3] Loose mix at time of construction.

commercial vehicles, respectively; similar data are not available for the other four test sections.

Initial plans called for obtaining information on the degradation of aggregates caused by the mixture's passage through the paver. Samples of loose mixtures both from the truck and from behind the paver were taken for this purpose. It soon became evident that normal experimental error in sampling and testing obscured such minor degradation as might have occurred. To reduce the effect of these sampling inconsistencies, a decision was made to average the aggregate gradations for the two loose mixtures of each test section. These averages are the gradations shown in table 1.

Aggregate Degradation

Table 4 shows, for the six test sections, the gradations of the aggregates in the loose mixture and in the pavement samples as determined by AASHO Method T 30, as well as the type of underlying structure, average daily traffic, and Los Angeles abrasion test wear loss. In addition, the table shows the computed values of surface area per pound of aggregate and the increase in surface area that may be attributed to degradation of aggregates caused by construction compaction and traffic. Aggregate surface area was chosen as a means of judging degradation because surface areas are sensitive to changes in grain size of aggregates. Degradation of aggregate is indicated by increasing values of surface area.

The aggregate surface areas shown in table 4 were computed according to the formula at the bottom of the table. The surface area factors of the formula were derived mathematically and are based on the factors used by California (*1*) [2] for a different set of sieve sizes. Surface areas reported in the table have values equivalent to those computed by the California procedure.

For simplicity, the data on degradation of aggregates occurring during construction and those on degradation occurring under traffic, as measured by changes in area, are tabulated in table 5. The data are grouped according to the type of underlying pavement structure, and within each such group arranged in the order of magnitude of their respective values of the Los Angeles abrasion wear loss.

Degradation from construction compaction

The increase in aggregate surface area under construction compaction with steel-tired rollers, as shown in table 5, indicates that the construction operation apparently caused degradation of the aggregate for all test sections. The increase in surface area was no more than 2 square feet per pound for five of the test sections and this is not considered significant. The increase in surface area of 6 square feet per pound of aggregate noted for test section 2 may be significant, however.

For the test sections constructed over rigid bases, the data show an average increase in aggregate surface area of 4 square feet per pound, but the average increase was only 2 square feet for those test sections constructed over semirigid or flexible bases. This is considered an indication that more degradation of aggregate occurs when pavement is constructed over rigid bases than over semirigid or flexible bases. This indication is strengthened by the fact that the grouped data show about the same averages for the Los Angeles abrasion test loss and for surface area of aggregate prior

Table 5.—Effect of construction compaction and traffic on degradation of aggregate

Type of base and section No.	Los Angeles abrasion test loss	Effect of construction compaction:[1] Aggregate surface area—			Effect of traffic				
		Prior to compaction	After compaction	Increase attributed to construction compaction	Traffic in test lane [2]	Age of pavement	Accumulated traffic [3]	Increase in aggregate surface area attributed to traffic [3]: For accumulated traffic	Increase in aggregate surface area attributed to traffic [3]: For first million vehicles
	Pct.	*Sq. ft./lb.*	*Sq. ft./lb.*	*Sq. ft./lb.*	*A.D.T.*	*Years*	*Million vehicles*	*Sq. ft./lb.*	*Sq. ft./lb.*
Flexible:									
Section 6	17	31	33	2	2,600	1.4	1	1	1
						2.8	3	2	
Section 1	32	42	44	2	400	1.4	------	0	1
						5.4	1	1	
Section 4	39	29	30	1	1,600	1.8	1	6	6
						4.6	3	6	
Average	29	34	36	2	------	------	------	------	------
Semirigid:									
Section 5	32	32	34	2	2,400	1.2	2	3	2
						4.0	5	2	
Rigid:									
Section 2	18	35	41	6	5,700	1.5	3	0	0
						5.4	11	1	
						7.9	16	1	
						9.1	19	0	
						12.0	25	2	
Section 3	39	31	33	2	3,000	4.2	5	1	1
						6.7	7	1	
						7.9	9	3	
						10.7	12	2	
Average	28	33	37	4	------	------	------	------	------

[1] Compacted with steel-wheel rollers.
[2] Approximate. See table 3.
[3] Rounded to whole numbers.

Table 6.—Density and change in density of asphalt pavement mixtures and pavement

Type of base and section No.	Age of pavement	Accumulated traffic [1]	Pavement density		
			Density of sample	Relation of pavement density to laboratory density	Increase in pavement density from traffic
	Years	*Million vehicles*	*Lb./cu. ft.*	*Percent*	*Lb./cu. ft.*
Flexible:					
Section 6	(2)	0	149.4	--------	--------
	0	0	144.3	96.6	--------
	1.4	1	150.1	100.5	5.8
	2.8	3	150.4	100.7	6.1
Section 1	(2)	0	134.7	--------	--------
	0	0	131.7	97.7	--------
	1.4	--------	136.2	101.1	4.5
	5.4	1	138.8	103.1	7.1
Section 4	(2)	0	141.5	--------	--------
	0	0	133.9	94.6	--------
	1.8	1	141.3	99.9	7.4
	4.6	3	142.5	100.7	8.6
Semirigid:					
Section 5	(2)	0	158.7	--------	--------
	0	0	155.4	97.9	--------
	1.2	2	160.5	101.1	5.1
	4.0	5	161.2	101.6	5.8
Rigid:					
Section 2	(2)	0	148.8	--------	--------
	0	0	147.4	99.0	--------
	1.5	3	150.6	101.2	3.2
	5.4	11	153.2	102.9	5.8
	7.9	16	152.7	102.6	5.3
	9.1	19	153 9	103.4	6.5
	12.0	25	154.0	103.5	6.6
Section 3	(2)	0	136.0	--------	--------
	0	0	135.7	99.8	--------
	4.2	5	141.6	104.1	5.9
	6.7	7	141.9	104.3	6.2
	7.9	9	141.4	103.9	5.7
	10.7	12	141.9	104.3	6.2

[1] Rounded to whole numbers.
[2] Laboratory samples of hot mixture from field, molded by compaction procedure of AASHO Method T167.

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[2] References cited by italic numbers in parentheses are listed on page 228.

to compaction. From a study of the data of table 5, it is obvious that the magnitude of the Los Angeles abrasion test loss cannot be related satisfactorily to degradation of aggregates caused by construction compaction as measured by increase in the surface area of the aggregates.

Degradation from traffic

The last group of columns in table 5 contains data on the degradation of aggregate caused by traffic, as indicated by increases in aggregate surface area. Data are provided on the estimated total number of vehicles that had passed over each test section prior to each sampling; the increase in aggregate surface area attributed to the traffic; and the increase in surface area after the passage of the first million vehicles, which was calculated by interpolation or extrapolation.

The increase in surface area of the aggregate attributable to traffic was negligible for all test sections except No. 4, which showed an increase of 6 square feet per pound after passage of 1 million vehicles.

All other conditions being comparable, the data show that traffic compaction caused slightly greater degradation of aggregates in pavements built over flexible bases than in pavement placed over rigid bases. It will be noted that averages of abrasion losses and of surface area of the aggregates prior to traffic for the pavements constructed over flexible bases are comparable to those for the pavements constructed over rigid bases. However, compaction from passage of 1 million vehicles caused increases in surface areas of 1, 1, and 6 square feet per pound, respectively, for the three test sections overlying flexible bases; but for the two sections overlying rigid bases, no increase in surface area was recorded for one and an increase of 1 square foot per pound was recorded for the other. Thus, as might be expected, degradation of aggregates resulting from traffic has a reverse relationship to the type of underlying structure from that of degradation caused by construction compaction.

The data in table 5 indicate no significant relationship between results of the Los Angeles abrasion test and the degradation of aggregate from either construction compaction or traffic. Aggregate degradation measured by changes in surface area was insignificant in most instances and had no apparent effect on pavement performance.

Pavement Densities

A comparison of the densities of the pavement mixture as originally determined for laboratory samples, as subsequently found for samples of the pavement after construction, and after varying periods of traffic service, is presented in table 6. Comparison of these data with those presented in table 5 reveals little or no relation between aggregate degradation and pavement densification.

Densities for laboratory specimens shown in table 6 were obtained from tests made on three or more 4- by 4-inch cylindrical specimens of hot field mixtures that were molded under a static load of 3,000 p.s.i. using the double plunger method (AASHO Method T 167 or ASTM Method D 1074).

The primary purpose of determining density in the laboratory was to provide a basis for checking the adequacy of construction compaction. Because the contractor was not requested to deviate from his regular procedure, the control of compaction was that normally required by the State of Maryland. In all instances, the density after construction compaction was essentially between 95 and 100 percent of the density of laboratory specimens. Section 4, with the lowest relative construction density, 95 percent, showed the highest increase in density from traffic compaction.

Construction compaction densities had a higher percentage of relationship to laboratory densities for the test sections overlying rigid bases than for those overlying flexible bases. This confirms the widely held belief that a firm base is more conducive to high density compaction of surfacing material than a yielding base. In all test sections, traffic compaction caused a considerable increase in density.

Test sections 2 and 3, constructed on rigid bases, appeared to have reached ultimate densities after 12 and 7 years of service and after passage of 25 and 7 million vehicles, respectively. This ultimate pavement density was approximately 104 percent of laboratory density. None of the other test sections appears to have reached ultimate density but, with the exception of section 1, none has been in service more than 5 years or has carried total traffic of more than 5 million vehicles.

Section 1 was constructed in the same year as section 2 but was surface treated soon after the 5.4-year period of sampling. The section was sampled a few times after application of this treatment but the test results were not comparable with those obtained from samples prior to treatment because of enrichment of the top portion of the pavement. This section was, therefore, discontinued as a part of the study after 5.4 years of service.

Air Voids

Pavement performance

The volume of contained air voids, which is a function of asphalt content for a particular aggregate, is an important factor affecting the performance of a dense-graded pavement. A low percentage of air voids in a pavement (high asphalt content) creates conditions conducive to rutting or shoving of the pavement or to surface bleeding. Too high a percentage of air voids (low asphalt content) creates conditions conducive to rapid hardening of the asphalt, which may result in early deterioration of the pavement from surface pitting, raveling, or cracking.

Table 7.—Properties of original and recovered asphalt compared with pavement age and percentage of air voids

Section No.[1] and source of sample	Air voids	Properties of original and recovered asphalt			
		Penetration	Retained penetration	Softening point	Ductility
	Percent		*Percent*		*Cm.*
Section 2:					
Original asphalt	--------	96	--------	115	205
Loose mix [2]	4.8	80	83	123	203
Pavement at age—					
0 years	5.7	--------	--------	--------	--------
1.5 years	3.6	78	81	121	210+
5.4 years	2.0	59	62	127	240
7.9 years	2.3	--------	--------	--------	--------
9.1 years	1.5	--------	--------	--------	--------
12.0 years	1.4	50	56	127	198
Section 5:					
Original asphalt	--------	96	--------	116	228
Loose mix [2]	3.6	77	80	120	210
Pavement at age—					
0 years	5.6	--------	--------	--------	--------
1.2 years	2.5	62	65	125	250+
4.0 years	2.1	54	56	128	250+
Section 6:					
Original asphalt	--------	91	--------	118	250+
Loose mix [2]	4.1	84	92	123	250+
Pavement at age—					
0 years	7.4	--------	--------	--------	--------
1.4 years	3.6	48	53	133	250+
2.8 years	3.5	35	38	139	173
Section 3:					
Original asphalt	--------	97	--------	113	200
Loose mix [2]	9.4	86	88	117	150+
Pavement at age—					
0 years	9.6	--------	--------	--------	--------
4.2 years	5.7	45	46	133	175+
7.9 years	5.8	29	30	142	138
10.7 years	5.5	30	31	139	98
Section 4:					
Original asphalt	--------	93	--------	118	165
Loose mix [2]	7.0	78	84	122	222
Pavement at age—					
0 years	11.9	--------	--------	--------	--------
1.8 years	7.1	41	44	133	198
4.6 years	6.3	30	32	139	64
Section 1:					
Original asphalt	--------	101	--------	120	250+
Loose mix [2]	12.6	--------	--------	--------	--------
Pavement at age—					
0 years	14.5	--------	--------	--------	--------
1.4 years	11.6	40	40	139	92
5.4 years	9.9	30	30	150	33

[1] Arranged in sequence according to air void content of most recent sample.
[2] Air voids shown are for laboratory molded specimens.

Table 7 and figure 1 show the effect of the percentage of air voids and pavement age on changes in ductility and retained penetration of the contained asphalt used in construction of the six test sections. The percentages of air voids were computed from the density data, recorded in table 6, and the maximum specific gravities of the mixtures shown in table 8. Data are also presented in table 8 on specific gravity and absorption for the aggregate portion of mixture; these data may be used, if desired, for computing the percentages of voids by other methods.

Table 8.—Specific gravity and absorption data

	Material from section—					
	1	2	3	4	5	6
Mixture: Maximum specific gravity [1]	2.469	2.504	2.406	2.437	2.638	2.498
Aggregate:						
Bulk specific gravity	2.59	2.73	2.57	2.55	2.87	2.68
Effective specific gravity [2]	2.69	2.74	2.64	2.64	2.88	2.72
Apparent specific gravity	2.76	2.78	2.66	2.67	2.92	2.72
Asphalt absorption, by weight ... percent [2]	1.5	.1	1.0	1.4	.1	.6

[1] By Rice vacuum saturation procedure (*2*) using loose mixture broken down from most recent pavement sample.
[2] Computed.

The data for the various test sections have been arranged in table 7 and figure 1 in sequence according to the magnitude of volume of air voids, as most recently determined. Thus section 2, with the lowest percentage of voids at the latest testing, is placed first.

The percentages of air voids for the laboratory molded specimens are also shown in table 7. The molding procedure followed for this study was that used by the Bureau of Public Roads in its current method of mix design (*3*) for wearing course mixtures. In this procedure, the asphalt content for a pavement to be subjected to light, medium, or heavy traffic is set at the value that will produce air voids of 6 percent in the laboratory specimen.

On the basis of this 6-percent air void criterion, tests on the molded laboratory specimens showed that the pavement mixtures for test sections 2, 5, and 6, with air voids of 4.8, 3.6, and 4.1 percent respectively, contained excess asphalt. To date, the pavement in these test sections has not shown evidence of excessive rutting or bleeding. However, incipient bleeding did develop in the pavement of test section 2 after about 6 years of service. The tests on the molded laboratory specimens from test sections 3, 4, and 1, which produced air voids of 9.4, 7.0, and 12.6 percent, respectively, showed that the pavement in these sections was deficient in asphalt. The pavement of section 3

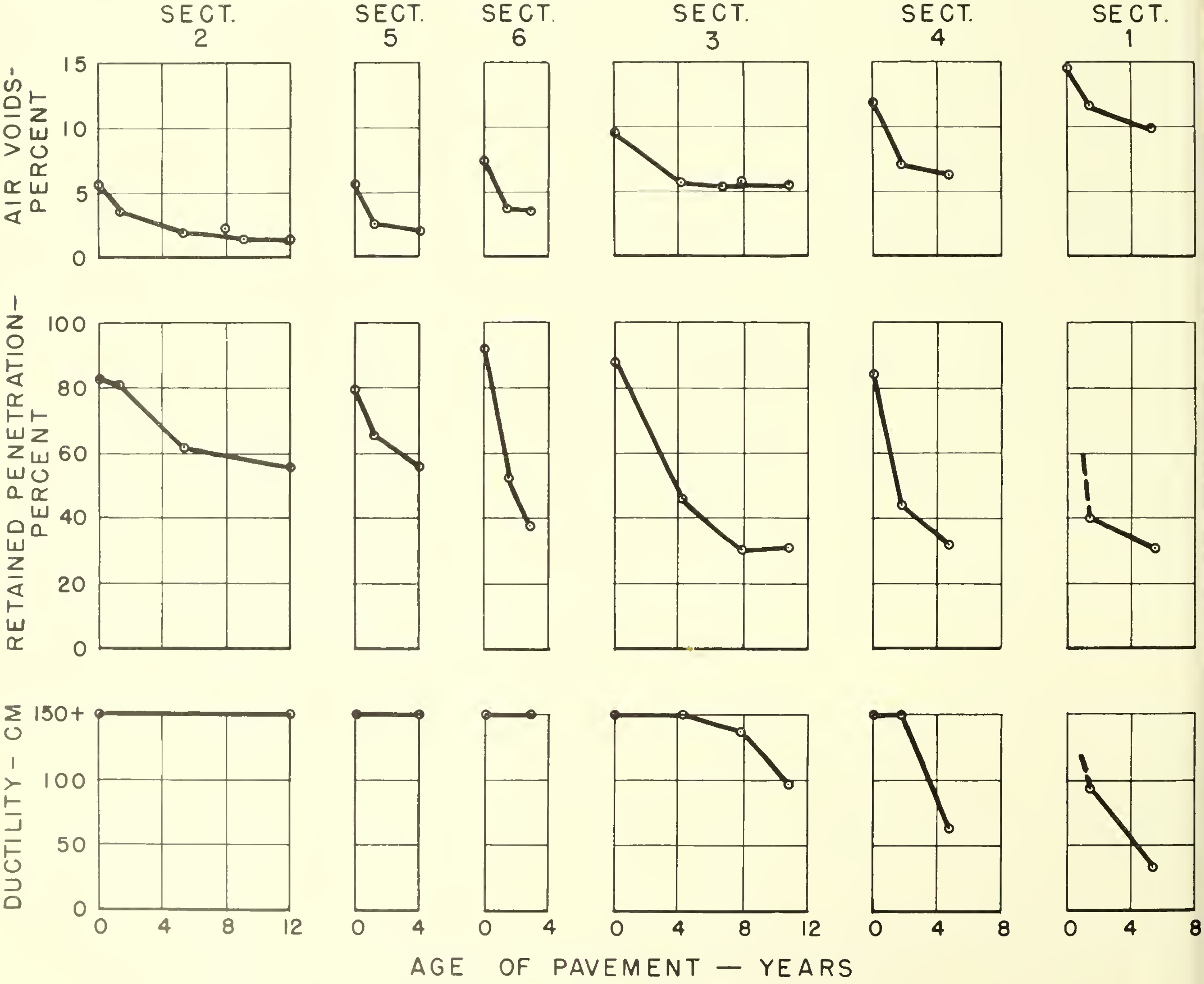

Figure 1.—Effect of percentage of air voids on age hardening of asphalt.

began to show evidence of slight surface pitting after 10 years of service and that of section 4 after approximately 2 years of service.

The pavement of test section 1 began to deteriorate rather severely early in its life. After about 6 years of service this pavement had developed such extensive hair cracking and surface pitting that a surface treatment was required. Based on the immersion-compression method of mix design, which accounts for asphalt absorption, the mixture for this pavement was deficient in asphalt by more than 2 percentage points.

The test data on pavement performance and the percentage of air voids obtained from tests on the laboratory specimens, tend to confirm the validity of the 6-percent air void criterion of the current mix design procedure used by the Bureau of Public Roads. The 6-percent value is greater than the 4.8-percent value of the laboratory specimen of section 2, which developed incipient bleeding; and less than the values developed for specimens of sections 3, 4, and 1, which showed distress in the form of pitting or cracking.

Asphalt hardening

The effect of the volume of air voids in the pavement on the rate of change in the characteristics of the contained asphalt also is shown in table 7 and illustrated in figure 1. The set of curves at the top of the figure are individual plots of air voids versus pavement age for each of the six test sections. The individual plots have been arranged in the same sequence of test sections as in table 7, as already described. The two sets of curves in the center and bottom of figure 1 are corresponding plots of retained penetration and ductility of the asphalt recovered from pavement samples. A combination of low penetration and low ductility is undesirable because it has been related to deterioration or impending failure of pavement in the form of cracking, pitting, or raveling (*4*, *5*). The rate at which such changes occur will be indicative of the length of satisfactory pavement performance.

Because all asphalts of the study were of an 85–100 penetration grade, the plotted values of percentages of retained penetration correspond very closely to actual values of penetration. From a comparison of the plotted results for retained penetration with those for air voids, it is evident that the percentage of air voids in the pavement affects the rate of hardening of the asphalt. Except for test section 6, the higher percentages of air voids in several test sections resulted in increasing rates of loss in retained penetration during the early life of the pavement. The results of the thin-film oven test (see table 2) indicated that the asphalt used for test section 6 was more susceptible to hardening than the other two asphalts subjected to this test; this might account for the faster hardening of asphalt in the pavement of section 6 than occurred in the pavement of test sections 3 and 4, which had higher percentages of air voids.

Comparison of the plotted results for ductility with the plotted percentage of air voids shows a very pronounced effect of the volume of air voids on changes in the ductility of the asphalt. It is noted that the asphalt ductility did not drop below 150 centimeters for the three test sections containing the asphalts with the lowest percentages of air voids. In the asphalts of the other three test sections (3, 4, and 1), the ductility fell below 150 centimeters; and the rate of decrease corresponded to the increasing percentage of air voids. The asphalt in test section 1, with the highest percentage of air voids, had the lowest ductility of any test section in this study, and it was this section that required surface treatment after approximately 6 years of service.

Figure 2 further shows the relation of the percentage of air voids in the pavement to the rate of change in retained penetration or hardening of the contained asphalt. In this figure, the percentage of initial air voids in the pavement is plotted against the retained penetrations of the asphalts after 4 years of pavement service. Retained penetrations were determined from the penetration values of the asphalt recovered from the loose mixture, after mixing; and from the values after 4 years of service, obtained by interpolation or extrapolation of the data from table 7. Figure 2 shows that, except for test section 6, the greater the percentage of initial air voids in the pavement, the faster the rate at which hardening of the contained asphalt occurs: For example, pavements in the test sections with a range in initial air voids of from 6 to 10 percent exhibited a range of from 72 to 49 percent in retained penetration.

From consideration of the data illustrated by figures 1 and 2 and shown in table 7, it may be concluded that a pavement with a volume of air voids appreciably greater than 7 percent at the time of its construction will have a relatively rapid rate of asphalt hardening.

The minimum percentage of air voids desired for pavement as compacted in construction, of course, depends on the probable percentage of air voids that will remain when the pavement has reached its ultimate density. If the percentage of air voids is too low at the time of construction compaction, eventual bleeding or loss of pavement stability will occur. The pavement of test section 2, which developed incipient bleeding, probably could not have tolerated an appreciable increase in asphalt; it was adequately compacted and had 5.7 percent initial air voids. It therefore appears that, for best assurance of satisfactory performance, a dense-graded asphaltic concrete pavement should have an air void volume of 5 to 7 percent immediately after construction, assuming adequate compaction. Because percentage of air voids is controlled primarily by the asphalt content for a particular aggregate, the limits previously described provide a good basis for making proper field adjustment of asphalt content based on the determinations of percentage of air voids in pavement samples after adequate compaction.

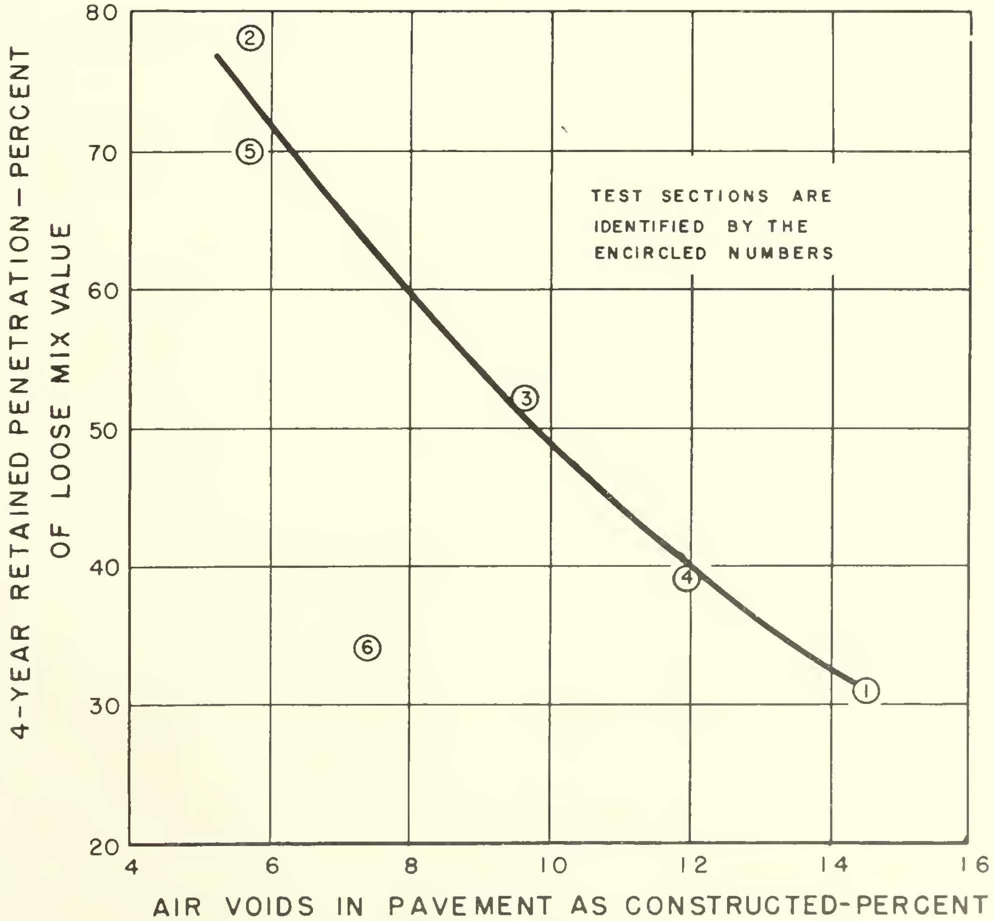

Figure 2.—Effect of initial percentage of air voids in pavement.

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New U.S. Highway Map

The Bureau of Public Roads has recently produced a 40- by 25½-inch map of the United States showing the routes of the National System of Interstate and Defense Highways, the Federal-aid Primary Highway System, and the U.S. Numbered Highway System. The map, printed in red, green, and black, is drawn to a scale of 1 inch equals 80 miles (1:5,000,000). The new map is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 25 cents per copy.

For the 41,000-mile National System of Interstate and Defense Highways, commonly called the Interstate System, the locations of all routes are shown on the map, but only about one-fourth of the mileage is open to traffic at present. The System is scheduled for completion by 1972. The Federal-aid Primary System totals about 224,000 miles (exclusive of the Interstate System); the great majority of its routes are parts of the State highway systems. The U.S. Numbered System, 170,000 miles in extent, was devised by the American Association of State Highway Officials as a means for guiding travelers, and does not designate Federal-aid highways. However, most U.S. numbered routes are on the road systems eligible for Federal aid.

It should be noted that while the map will serve a variety of useful purposes, it is not a touring or road-condition map.

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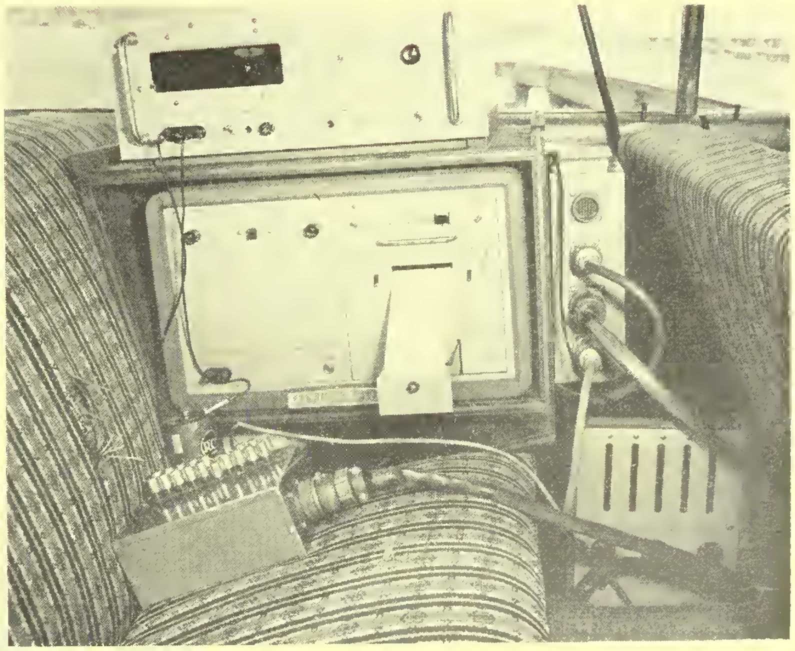

A digital recording instrument installed on the back seat of a rear test car, and two cars with a communications system installed on the lead car, which were used for the study on intervehicle spacing. *The Effect of Speed Change Information on Spacing Between Vehicles*, appearing in this issue.

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U.S. DEPARTMENT OF COMMERCE
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BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

The Effect of Speed Change Information on Spacing Between Vehicles

BY THE TRAFFIC OPERATIONS DIVISION
BUREAU OF PUBLIC ROADS

Reported [1] by RICHARD M. MICHAELS, Research Psychologist,
and DAVID SOLOMON, Highway Research Engineer

Variations in the spacing maintained between vehicles has been attributed primarily to the drivers' reactions to changes in the speed of the preceding vehicle. On this basis, tests were conducted, with two- and three-car queues, in which a communications system was employed to transmit advance speed change information to the driver of a rear test car. Test data were analyzed to determine the effectiveness of the communications system—the test drivers' responses were studied to determine whether intervehicle spacing had been modified as the result of transmission of the advance speed change information.

This study was undertaken because intervehicle spacing is considered to be one of the more important factors affecting the stability of car-following patterns and thereby influencing the volume of traffic that can be moved on a highway. Analysis of the two-car test data indicated that use of a communications system, similar to the one employed in the study reported here, could increase the traffic-carrying capacity of a highway. From analysis of the three-car test data, it was noted that most of the potential effect of the communications system had been eliminated by interposition of a third car. With use of the communications system, a relation between advance warning time and the distance headways was noted for two-car queues.

Variability in headways was substantially reduced when the driver of a rear car had advance information on the speed changes. The variability noted in driver responses, to information provided by the communications system, indicates that the stability of intervehicle spacing is greatly influenced by fluctuations in the individual driver's psychological state.

Introduction

RECENT RESEARCH (*1*) [2] indicates that variations in spacing between vehicles in a queue are determined primarily by the drivers' reactions to changes in speed of the vehicle ahead and, secondarily, by the distance between vehicles. When a change in speed is imposed in the lead car, the driver of the second car, in order to maintain his spacing, reacts to the change in speed—not the change in distance. Thus, he changes the speed at which his car is traveling to eliminate any difference in speed between the cars. The stability of the operation depends therefore upon the length of time elapsing from the lead vehicle's change in speed to its detection by the driver of the following car, as well as his ability to modify the speed of his car.

As pointed out by Brown (*2*), estimates of speed changes ultimately depend on the driver's ability to detect changes in visual angle (the angle, at the eye of the observer, subtended by the boundaries of an object), which governs the time a driver requires to detect changes in the speed of the lead vehicle. Because of human limitations, small changes in visual angle and hence in speed are not detectable. Consequently, in normal car-following patterns a time lapse occurs between the initiation of a speed change in the leading car and its detection by the driver of the following car. To a large extent, much of the instability of the queues as demonstrated by other car-following studies could have resulted from the relative insensitivity of the drivers to small changes in visual angle.

Also contributing to variance in spacing of cars in a queue is the fact that the driver of a following car has no prior knowledge of the occurrence of any acceleration or deceleration of the preceding vehicle nor the magnitude of that change of speed. (Display of brake lights or hand signals may warn of deceleration.) Therefore, the following driver may be expected to adopt a spacing to compensate for his uncertainty stemming from these two factors: his inability to detect small changes in visual angle and his inability to anticipate the change in speed.

Two methods for improving the stability of car-following are suggested by consideration of the foregoing analysis: (1) To improve the drivers' abilities to detect changes in visual angle, or (2) to provide the drivers of following cars with information about changes in the speed of the lead vehicle before such changes are initiated. The second method provided the basis for the study discussed in this article.

Possible predictions on driver behavior

From the foregoing analysis several hypotheses are possible about the behavior of the driver of a car, when he is given information on the type and magnitude of each change in speed to be made in the lead vehicle.

First, the driver of the following car, by responding to this advance information, should be able to compensate for changes in speed in the lead vehicle by beginning his maneuver prior to the onset of the change. The degree of compensation effected should depend upon the interval of time between the driver's receipt of the information and the onset of the speed change in the lead vehicle.

Second, because the driver of the following car does not have to rely on his ability to detect changes in speed, he will not have to maintain a spacing commensurate with his maximum ability for such detection. Consequently, it is reasonable to assume that the average headway during a constant speed operation would be less with a communications system than without it. (Headway is the time or space interval between two vehicles traveling the same route. For precise reference, the terms distance headway and time headway are used in this article.)

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[1] This article was presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] References cited by italic numbers in parentheses are listed on page 235.

Third. The variance in headway between cars in a constant speed operation should be decreased when communication is employed.

An attempt was made in this study to determine whether, in a simple pattern of one car following another, intervehicle spacing would be modified when drivers were provided advance information on speed changes.

Summary and Conclusions

From this study with two- and three-car queues, it was determined that the effect of the use of the communications system on the spacing between two vehicles varied with the speed at which they were moving; its use had little effect when the speed was 36 m.p.h. (miles per hour) but resulted in a large reduction in distance headway when the speed was 54 m.p.h. However, when another vehicle was interposed, making a three-car queue, the communications system's effect on distance headway was nearly eliminated at all speeds. During constant speed operations, at any of the three speeds used, the variance in headways between the two cars was significantly reduced by the communication of speed change information to the test drivers. The transmission of this information approximately 3 seconds prior to initiation of the speed change by the lead car permitted maintenance of the most uniform headways.

On the basis of data collected during the study, it is concluded that communication of advance speed change information from one driver to another is an extremely complex problem. Further, no simple or direct means exists for transmitting advance speed change information that will result in a universal or predictable modification of headway patterns in car following.

Recording Instruments and Their Use

A stadimeter (a range-finding device) was used in this study to measure the distance headway between a pair of vehicles. The stadimeter operates on the same principle as the sextant in that two images are brought into coincidence by use of a rotating mirror. Because use of the stadimeter requires specification of some dimension of the target whose range is sought, the dimension was marked by two brightly painted targets mounted on the rear bumper of the lead vehicle (fig.1). The mounting of the stadimeter in the second car is shown in figure 2.

Operation of stadimeter

The stadimeter operator viewed the lead car through the ring sight on the right of the instrument. While looking at one target, he slowly rotated the mirror on the left of the sight by turning the crank, on the left side of the instrument, until the image of the second target coincided with that of the first. The hand crank also was geared to drive a precision potentiometer so that the angular displacement of the mirror was translated into a voltage change. The distance in feet between the vehicles is inversely related to the angular rotation of the mirror. By use of a calibration procedure the voltage change was converted to the distance in feet.

The task of the stadimeter operator was to keep the two targets in coincidence during the run. For distances of more than 50 feet, tracking error generally was less than 5 percent, which was adequate for purposes of this study. A digital recording system, described in a report by Hopkins (*3*), was employed to store the data. Headway readings were converted to digital form by use of a digital voltmeter, were read out, and were stored by use of a digital printer. The speed of the rear vehicle also was determined, digitized, and stored in the printer. An equipment operator in the rear car manually coded the following listed data into the printer: Each speed change made by the lead vehicle, its speed at the beginning and ending of each maneuver, and the time each maneuver was started and ended.

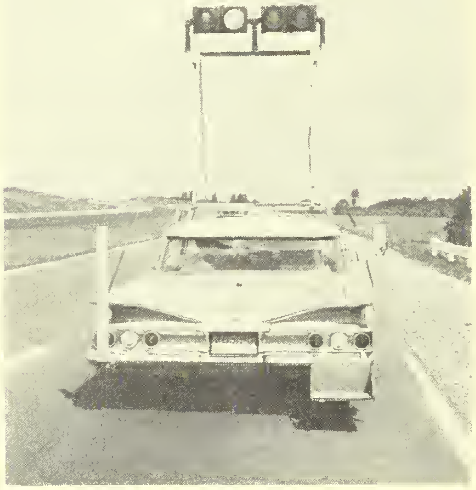

Figure 1.—Lead car with overhead signals and targets on rear bumper.

By means of radio equipment installed in the two vehicles, the lead car observer could notify the equipment operator in the following car what change of speed was to occur and when the change began and ended. This was done only when no visual communication signal was given. As the equipment operator used head phones, the driver of the following car could not hear these messages.

Four rates of acceleration were used for each series of maneuvers: +1, +3, −1, and −3 m.p.h./sec. (miles per hour per second). The interval between each maneuver in a series was either 15, 30, or 45 seconds in duration. An automatic recycling timing system had been installed in the lead vehicle to specify the individual maneuver to be undertaken, the advance warning time (if any) to be signaled to the following car, the duration time of the maneuver, and the interval between the beginning of each maneuver in the series and the beginning of the next. By means of cue lights on the dashboard, the driver of the lead car was notified when to begin and end each maneuver. In general, the driver's control in making speed changes was accurate to within 5 percent.

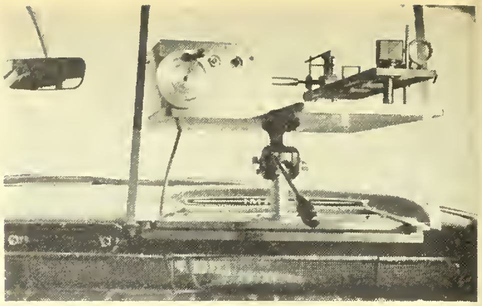

Figure 2.—Stadimeter mounted in rear car.

The information display for the driver of the following car consisted of four traffic signal units with white lenses, 8 inches in diameter, mounted above the lead vehicle, as shown in figure 1. Each signal unit represented one of the four possible speed change maneuvers. The size of the signal units and the height at which they were mounted above the lead vehicle were dictated by possible conditions of grade, curvature, and length of queue that might be employed for any particular study to be made with this display.

Site of Study

A 7-mile portion of Interstate Highway 70, located between Gaithersburg and Rockville, Md., and considered typical of a rural freeway, was selected as the study site. This freeway is a four-lane divided highway on which there is full control of access. Each of the four lanes, constructed of bituminous concrete, is 12 feet wide. Although the shoulders generally are 10 feet wide (8 feet of gravel and 2 feet of grass), a small part of the study site has gravel shoulders 12 feet wide. The dividing median is a grass strip 50 feet wide.

The speed limit at the test site is 60 m.p.h. and only a few vehicles were observed exceeding this limit. Typical of a rural freeway, the design speed for this section of the highway is 70 m.p.h.; maximum gradient is 3 percent, except for a length of 1,000 feet where the grade is 3.5 percent; and the maximum horizontal curvature is one degree.

The daily traffic volume on the section of the highway used for this study was about 10,000 vehicles. While the study was underway, daytime traffic averaged about 600 vehicles per hour or about 300 vehicles per hour on each one-way roadway—a relatively low volume of traffic for a freeway near a large metropolitan area. This low traffic volume and the high-type design characteristics of the highway were the principal reasons for selection of this study site. These factors made it possible to drive at a reasonably wide range of speeds for the required length of time with little interference to or from other traffic.

Procedures

Either two or three vehicles were used in the test runs for this study in which headways were measured for the rear cars, which were driven by test drivers. In addition to the test driver, the rear car carried the stadimeter operator (observer) and the equipment operator.

Drivers

Seven drivers were used for this study. Four were summer employees of the Bureau of Public Roads; their ages ranged from 18 to 24 years; all had had at least 2 years of driving experience. The other test drivers were the experimenters conducting the study. All seven drivers participated in the test runs made with the two cars but only the four test drivers participated in the three-car test runs. To isolate any effects of possible bias, the primary analyses of the data collected from two-car tests were made only for runs by the four test drivers. Because analyses of the data indicated responses of all seven drivers to be quite comparable, information collected from all two-car tests was used for some of the subsidiary analyses.

Before the beginning of each test run, each test driver was given the following instructions: "Assume you are driving in rush hour traffic. Assume that vehicles are in the left lane beside you as well as behind you. Drive as you would in this type of traffic by keeping pace with the vehicle ahead so as to minimize the possibility of other vehicles weaving in front of you."

Two-car studies

In the test procedure with two cars, the speedometer of the rear vehicle was covered and the tail lights of the lead vehicle were disconnected for all test runs. The lead car carried out precise maneuvers and the headway was measured between the two vehicles.

Before the beginning of each test run, the study's signal system was explained to the test driver. He was told that when the signal lamp on the extreme left of the lead car was lighted a fast acceleration would be made, and that the next three signals would indicate fast deceleration, slow acceleration, and slow deceleration, respectively. The signal lamp was lighted prior to the onset of the speed change for a period of time determined by the preselected warning condition and remained lighted until completion of the maneuver.

A run included 16 speed changes—four repetitions each of maneuvers from 36 to 45 m.p.h., from 45 to 36 m.p.h., from 45 to 54 m.p.h., and from 54 to 45 m.p.h. Just prior to the start of each run, the test driver was also informed as to which of the five test conditions would be used: without communications or with communications at one of the four warning times (0, 1, 3, or 5 seconds). For the "without communications" condition, the signal lamps were not used and the test driver received no information concerning the maneuver to be executed. When the run was made with communications, the signal system was used to indicate the magnitude of the speed change to be made during each maneuver (+1, +3, −1, or −3 m.p.h./sec.), as well as to show the preselected warning time conditions.

All runs were started when both vehicles were traveling at a speed of 45 m.p.h. The programing device was then activated in the lead vehicle and one of the four preprogramed speed changes was presented on the appropriate signal lamp. At the end of the first maneuver of each run, the lead vehicle was traveling at a speed of either 36 or 54 m.p.h. Therefore, speed changes for the second maneuver of a run required acceleration from 36 m.p.h. or deceleration from 54 m.p.h. Thus, the test driver's uncertainty as to the speed change that would be presented was only half that he experienced when the next maneuver was to be made from a speed of 45 m.p.h.

At the end of each run, another warning time condition was randomly selected; the test driver was informed of the new condition; and another run was executed by the two cars. This procedure was repeated until the test driver had completed five runs in succession, one for each warning condition. Each of the seven test drivers made five successive runs and the entire procedure was repeated four times so that each driver completed four runs for each of the five test conditions.

Three-car studies

In the three-car studies, the signals for indicating speed changes and warning time were mounted above the first vehicle, which initiated all maneuvers, but the targets for headway measurement were mounted on the rear of the second car. The test driver operated the third car. Speedometers of both the second and third cars were taped over. The procedure for measuring the intervehicle spacing between the second and third cars was the same as that used for test runs with two cars. Runs were made with only three test conditions rather than five: without communications and with communications during which the warning time was either 1 or 5 seconds. Only the four test drivers were used for tests with the three cars and test runs for each of the three conditions were repeated by each driver three times. The drivers of the second and third cars were given instructions similar to those given for the two-car runs.

ANALYSES

Choice of Headway at Constant Speed

Four independent variables were studied in the analysis of headway: (1) Speed, (2) driver, (3) run, and (4) type of communications. Data were obtained for all 240 combinations of the four variables in the case of the two-car following situation. For the three-car data analysis, the 0-second and 3-second warning time conditions were omitted.

Information was obtained from all possible combinations of the independent variables for the time intervals between maneuvers; that is, when the two vehicles were traveling at some constant speed. Samples of information, consisting of the record of speed and headway of both vehicles, were taken at one-second intervals by the digital recording system. To eliminate any time coherence among the sample points, additional samples were taken from recorded data at five-second intervals. In general, 10 to 15 samples were taken from each of the two speeds of 36 and 54 m.p.h., which were maintained during four periods of time in each test run, and 20 to 40 samples were taken during the eight periods of time the speed was constant at 45 m.p.h. In order to obtain a completely balanced block design for the analysis of variance, a final sampling of 10 random observations was made for each of the three speeds of each run.

Analysis of Variance Data for Two-Car Runs

An analysis of variance was performed on the two-car data. Four main variables of three speeds, four drivers, four runs, and five communications conditions were used. For each of the 240 combinations of these variables, 10 headway measurements were employed, thus making 2,400 headway observations. The analysis demonstrated that not only were the main variables significantly different at the 0.01 level, but also that all

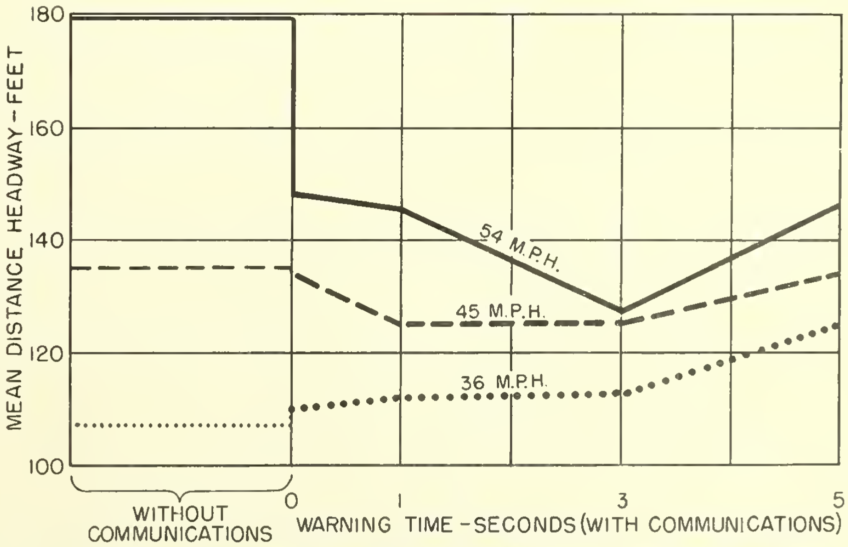

Figure 3.—Mean distance headways maintained in two-car runs without and with communications.

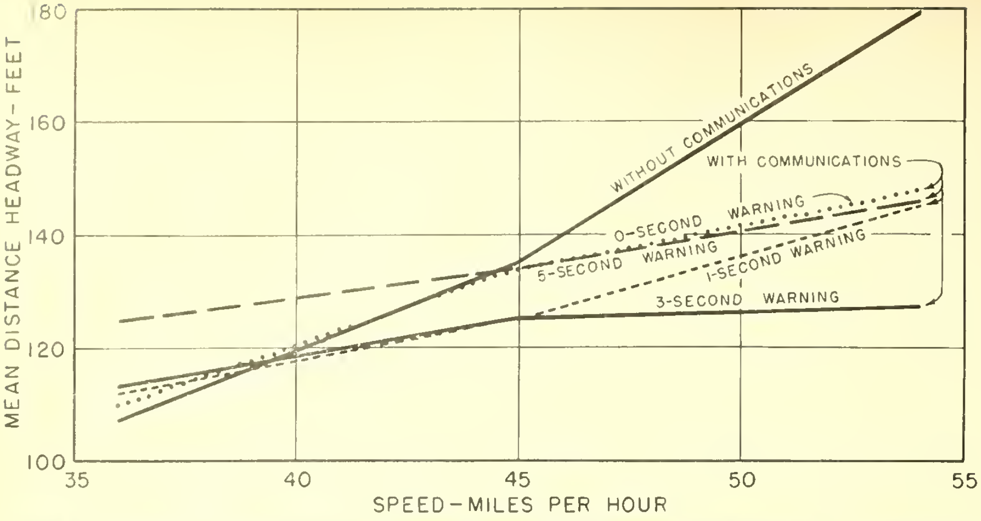

Figure 4.—Effect of warning time on mean distance headways in two-car runs at each test speed.

six first order, four second order, and the third order interaction terms were significant at the 0.01 level. As a check on the sampling procedure, an additional analysis of variance was performed on all the variables for one run (speed, driver, type of communications), and identical results of the analysis were obtained: all terms were significant at the 0.01 level.

Interaction of speed and warning time

For more detailed analysis of the effects of the communications system, data were separated on the basis of speed. At the highest speed of the runs, 54 m.p.h., consideration of the data reveals a very significant reduction in mean distance headways when the communications system was used. This information is graphically presented in figure 3. With communications, the mean distance headways varied from a minimum of 127 feet to a maximum of 148 feet; without communications, the mean headway was 179 feet. True headway distances (from the rear end of the lead car to the rear end of the rear car), used for this analysis of the two-car test data, were obtained by adding a constant of 10 feet to the recorded headway, which the stadimeter had measured from the dashboard of the following vehicle to the rear bumper of the leading vehicle. At the lowest speed used in the study, 36 m.p.h., headways were no shorter with the communications system than without it. At the intermediate speed of 45 m.p.h., the headways began to decrease with use of the communications system.

Effect of warning time

Warning times had a considerable effect on the headways maintained, as shown in figure 3. With a 3-second advance warning time at 54 m.p.h., the drivers maintained minimum headways. At 45 m.p.h. minimum headways appeared to have been maintained within the range of a 1- to 3-second advance warning time. But at a speed of 36 m.p.h., no differences in headway occurred when the warning time was less than 3 seconds. However, with the longest advance warning time of 5 seconds, headways maintained at all three speeds were much longer than those maintained with the 3-second warning time. At 54 m.p.h., headways maintained with a 5-second warning time were about the same as those maintained with a 1-second warning time and were 15 percent longer than those maintained with a 3-second warning.

In figure 4, the relationship of headway to speed is illustrated with warning time as the parameter. Without communications, distance headways increased sharply as speed increased. For each of the four test run conditions with communications, distance headways increased less as the speed increased. Most noteworthy is the fact that the shortest headways were maintained with the 3-second warning time and, more importantly, were practically independent of speed. Therefore, 3 seconds may be considered the optimum warning time.

A further comparison of the relation of speed to headways maintained was made by converting the distance headways (fig. 3) to time headways for each speed range. These respective time headways were plotted against speeds for each test condition. As shown in figure 5, when maneuvers were performed without communications the time headways increased at the highest speed: but for each of the four warning-time conditions with communications, the time headways decreased with increased speeds. The lowest value of time headway was obtained from the data on the 3-second warning time at the speed of 54 m.p.h. Note the relation of time headway shown (fig. 5) for the condition without communications and that shown for the 3-second warning time. At a speed of 36 m.p.h., the difference in time headways shown for the two communications conditions is not statistically significant, but at speeds of 45 and 54 m.p.h. the time headways obtained with a 3-second warning time were significantly shorter than those from the condition without communications. At a speed of 54 m.p.h. this reduction was 30 percent.

Summary of Tabulated Data

A tabular summary of all data collected during this study for the two-car runs is shown in table 1. Each entry represents the average of 40 observations of data collected for all four runs averaged together.

The time headways for this study (shown in table 1) were calculated from the distance headways, as noted previously. For the calculation, it was assumed that the lead and

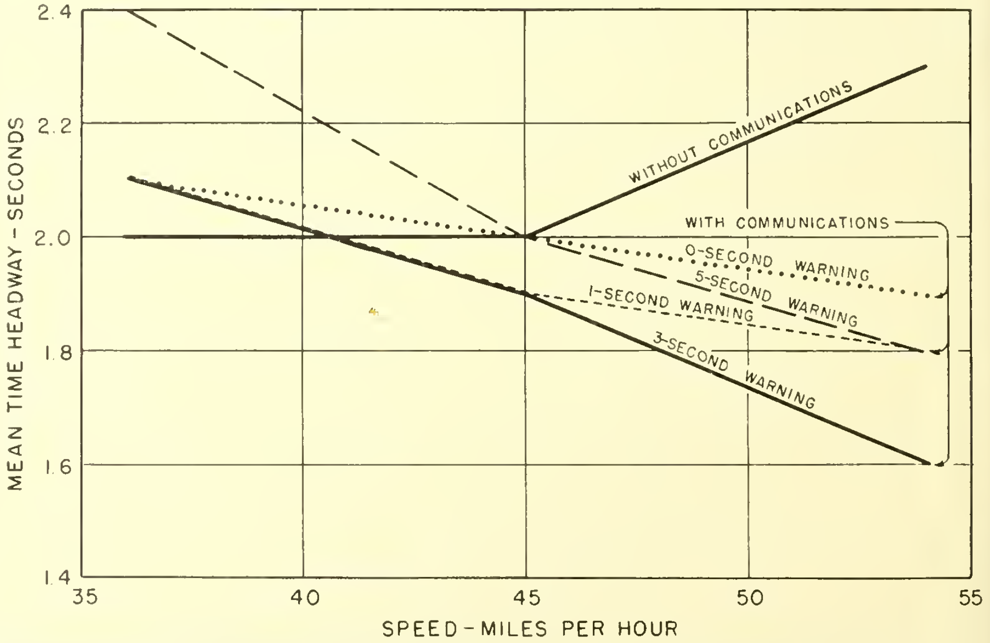

Figure 5.—Mean time headways maintained in two-car runs with communication of several warning times.

rear vehicles were traveling at an identical speed of either 36, 45, or 54 m.p.h. Because the drivers of the lead cars were extremely careful to maintain precise speed relationships, this assumption was close to the actual situation.

Table 1.—Mean distance and time headways maintained by each of four drivers at three speeds with and without communications in two-car runs [1]

Warning time	Driver A		Driver B		Driver C		Driver D		Average	
36 MILES PER HOUR										
Seconds	*Feet* [3]	*Seconds*	*Feet* [3]	*Seconds*	*Feet* [3]	*Seconds*	*Feet* [3]	*Seconds*	*Feet*	*Seconds*
None [2]	92	1.7	113	2.1	116	2.2	106	2.0	107	2.0
0	111	2.1	120	2.3	111	2.1	97	1.8	110	2.1
1	116	2.2	108	2.0	130	2.5	94	1.8	112	2.1
3	114	2.2	116	2.2	127	2.4	94	1.8	113	2.1
5	132	2.5	132	2.5	134	2.5	102	1.9	125	2.4
45 MILES PER HOUR										
None [2]	114	1.7	165	2.5	149	2.3	112	1.7	135	2.0
0	131	2.0	173	2.6	124	1.9	110	1.7	134	2.0
1	122	1.8	127	1.9	131	2.0	119	1.8	125	1.9
3	114	1.7	136	2.1	142	2.2	108	1.6	125	1.9
5	129	2.0	152	2.3	148	2.2	107	1.6	134	2.0
54 MILES PER HOUR										
None [2]	155	2.0	224	2.8	182	2.3	157	2.0	179	2.3
0	142	1.8	189	2.4	131	1.7	129	1.6	148	1.9
1	128	1.6	176	2.2	144	1.8	136	1.7	145	1.8
3	117	1.5	137	1.7	143	1.8	112	1.4	127	1.6
5	120	1.5	189	2.4	156	2.0	118	1.5	146	1.8

[1] Headways in two-car runs were measured from rear end of lead car to rear end of rear car.
[2] None indicates no communication at all.
[3] Indicates average of four runs.

A large variability in response of the four drivers both to speed and the type of communications employed is evident from consideration of the data. Because of these significant differences in driver reactions, the averaged data shown in the summary column of table 1 should be interpreted with extreme caution; these data, of course, represent a combination obtained from different populations.

As noted previously and as shown in figure 4, the mean distance headways when the warning time was 3 seconds were practically the same at all three speeds—between 113 and 127 feet; this range of only 14 feet is also shown in table 1. The range in mean distance headways over the three speeds for the other four communications conditions varied from 21 to 72 feet; generally, the same range was noted in the headways maintained by each driver. Test data also revealed that, among the four drivers, the relationship of mean time headways noted for conditions of no communications and the 3-second warning time is consistent (table 1). Although the absolute values of the time headways for the four drivers differ significantly, the change in time headway as a function of speed is similar in that the time headways for the 3-second warning time decreased sharply as the speed increased.

Communications Markedly Reduce Variation in Headway

Data collected from runs made by the three experimenter drivers were added to the data for runs by the four test drivers for a further analysis of the variation in headways. Standard deviations of distance headways were calculated for each of the three speeds under two test conditions, without communications and with the optimum warning time of 3 seconds. These calculations are shown in table 2. When data from the no communications condition and the 3-second warning time communication condition were compared, a reduction in standard deviation was noted for 19 of the 21 comparisons, and only one increase in standard deviation occurred. In most cases, the reduction was substantial and in 15 of the comparisons the differences were statistically significant. The median ratio of the standard deviation over all three speeds for all seven test drivers was 0.57, a reduction of 43 percent. In other words, communications reduced the variation in headway very substantially.

Analysis of Speed Changes During a Maneuver

Another analysis of data obtained from test runs with two cars was made concerning the speed changes carried out by the test drivers during the maneuvers. Although the limitations of the data processing system prevented a precise analysis of the time course of the changes in speeds, it was possible to determine some of the effects of the communications system by the control responses of the test drivers. An example of typical responses is shown by the curves in figure 6. When the speed change for the lead car was a slow acceleration from 45 to 54 m.p.h. at the rate of 1 m.p.h./sec., the test drivers tended to respond so that the position of the rear vehicles changed from leading to lagging as the warning time decreased from 3 seconds to 0 seconds, to a condition of no communications.

This analysis also revealed some indication that the test or rear car drivers were using the warning intervals to begin the speed changes; such a response is shown clearly by the curve in figure 6 for the 3-second warning time. The drivers of the rear cars had closed on the lead vehicles during the initial part of a maneuver, when the optimum warning time of 3 seconds was employed, then at some point toward the end of the maneuver had eased up on the accelerator thereby increasing the distance headways. The deceleration at the end of the maneuvers tended to show a great variability among the drivers and from one trial run to another.

Analysis of Data for Three Cars

For the portion of the study made with three cars, data on the effect of speed change information on intervehicle spacing were collected from only three test runs made by each of the four drivers.

For three cars, the headways were measured from the rear of the second car to the rear of the third car; communications were transmitted from the lead car. The third or rear car was driven by a test driver.

An analysis of variance also was carried out on the data collected from test runs of the three cars. With one exception, all terms were significant at the 0.01 level; the third order interaction term was significant at the 0.05 level.

Table 2.—Standard deviations of distance headways and their ratios for each of seven drivers at three speeds in two-car tests

Driver	36 miles per hour—			45 miles per hour—			54 miles per hour—		
	Without communications	With communications	Ratio	Without communications	With communications	Ratio	Without communications	With communications	Ratio
	Feet	*Feet*		*Feet*	*Feet*		*Feet*	*Feet*	
A	16	16	1.00	24	21	0.88	24	15	[1]0.63
B	21	14	[1].67	25	17	[1].68	27	13	[1].48
C	78	16	[1].21	67	16	[1].24	67	24	[1].36
D	68	28	[1].41	56	30	[1].54	60	16	[1].27
E	15	10	[1].67	18	24	1.33	20	17	.85
F	11	9	.82	18	16	.89	40	11	[1].28
G	14	8	[1].57	26	12	[1].46	29	13	[1].45
Median			[1].67			[1].68			[1].45

[1] Significantly different from 1.00 at the 0.05 level or less.

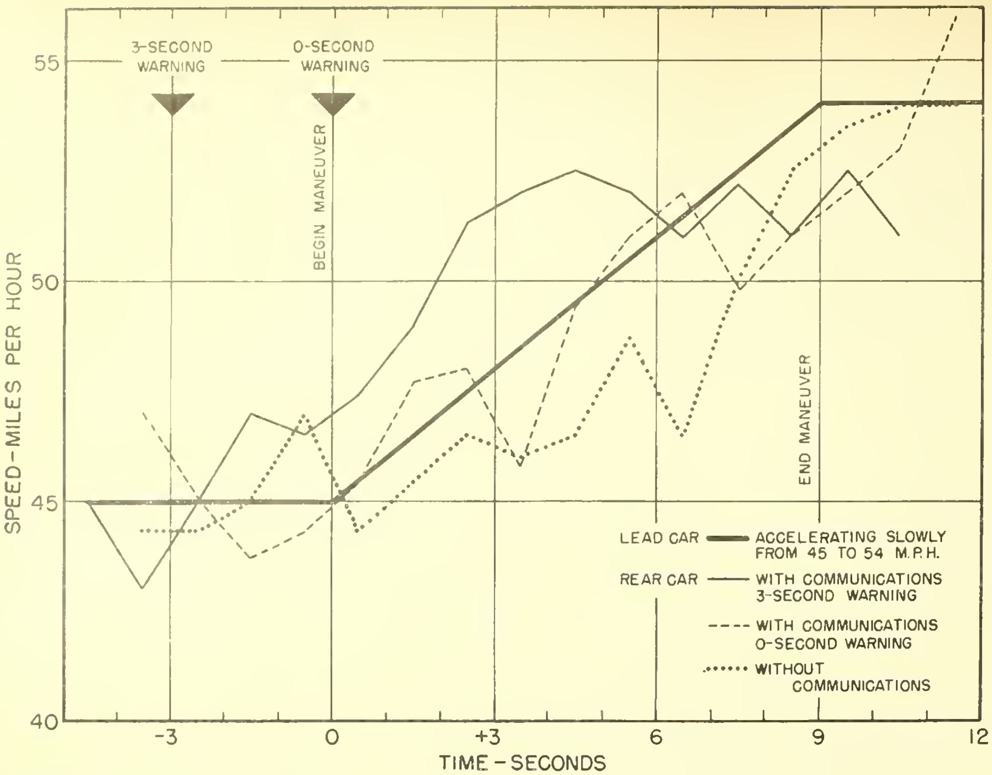

Figure 6.—Example of test driver responses during a maneuver in two-car runs. (Note anticipatory response for the 3-second warning time.)

Analysis of three-car data showed that the mean headways were considerably less than those calculated for two cars, as can be seen by comparison of information detailed in table 3 with that shown in table 1. In general, time headways were about one-half second shorter for all communications conditions in three-car test runs. Moreover, the influence of the communications system was far less effective in causing the drivers to reduce following distance than it had been during the two-car runs.

With three cars and all three communications conditions, a small increase was noted in the mean distance headways as the speeds increased from 36 to 54 m.p.h. However, decreases in mean time headways occurred both with and without communications; this decrease in time headways as speeds increased was larger when the test runs were made with communications. Table 3 contains headway data collected for three-car test runs.

When the interposition of a vehicle between the lead car and the rear car is considered, the 1-second warning time for the three-car test runs appears to have had an influence on the headways maintained by the test drivers that is probably equivalent to the influence of the 3-second warning time condition during two-car test runs. From examination of the test data presented in the summary column of table 3, it can be seen that with the 1-second warning time a range of only 16 feet in distance headways occurred for three-car runs between the speeds of 36 and 54 m.p.h. When the 5-second warning time condition was employed the range in headways increased to 24 feet, and when the condition was without communications it increased to 34 feet. The 16-foot minimum range with the 1-second warning time is consistent with the lowest range in distance headways of 14 feet recorded at all three speeds for two-car tests with a 3-second warning time condition.

Table 3.—Mean distance and time headways maintained by each of four drivers at three speeds with and without communications in three-car runs [1]

Warning time	Driver-- A		Driver-- B		Driver-- C		Driver-- D		Average	
	36 MILES PER HOUR									
Seconds	*Feet* [3]	*Seconds*	*Feet* [3]	*Seconds*	*Feet* [3]	*Seconds*	*Feet* [3]	*Seconds*	*Feet* [3]	*Seconds*
None [2]	78	1.5	73	1.4	73	1.4	91	1.7	79	1.5
1	90	1.7	71	1.3	97	1.8	88	1.7	87	1.6
5	97	1.8	85	1.6	77	1.5	99	1.9	90	1.7
	45 MILES PER HOUR									
None [2]	92	1.4	100	1.5	84	1.3	107	1.6	96	1.5
1	99	1.5	84	1.3	92	1.4	107	1.6	96	1.5
5	110	1.7	103	1.6	89	1.3	103	1.6	101	1.5
	54 MILES PER HOUR									
None [2]	107	1.4	113	1.4	96	1.2	136	1.7	113	1.4
1	109	1.4	97	1.2	91	1.1	116	1.5	103	1.3
5	104	1.3	130	1.6	93	1.2	128	1.6	114	1.4

[1] Headways in three-car runs were measured from rear end of second car to rear end of third car.
[2] None indicates no communication at all.
[3] Indicates average of three runs.

Discussion of Data Analyses

Analyses of the data collected during this study indicate that substitution of an alternate information path for judgment of speed changes is possible, and that drivers are able to utilize a very simple coding scheme (communications system), at least in a two-car following situation. In other words, the driver of a car generally adapts his following distance according to the advance information he has about the changes in speed to be made by the lead car and his ability to make corresponding compensatory control changes in the speed of his own vehicle.

The major contribution of the signal system was its provision of information that permitted the test driver to begin his compensatory speed change before the onset of the speed change in the lead vehicle. Thus, a driver used the actual change in speed of the lead vehicle as a feedback on his speed control responses. This is illustrated by the curves in figure 6.

Assistance from the communications system received by the test driver was least during an acceleration maneuver. Although limitations in the recording system precluded a complete analysis of the total effect of the communications system, some indication was noted of a lower rate of acceleration or deceleration with the display of speed change information than without it. However, considerable variation in acceleration or deceleration occurred during individual maneuvers, and this apparently resulted from the necessity for a driver to make visual angle discriminations and, more importantly, the predictions of future speed of the vehicle preceding his own. The experimental system employed in this study, of course, added nothing that a driver could use for such purposes.

The warning time—that is, the interval between display of the visual signal and the initiation of the speed change of the lead vehicle—was also observed to be an important factor in the following pattern adopted by the test driver in the rear car during constant speed operations. Analysis of the data showed that, dependent on the speed at which the test cars were traveling, minimum headways and minimum variance were obtained when the interval of warning time was between 1 and 3 seconds. The warning time obviously was not employed solely as a reaction time but rather was used as a total response time of the system, encompassing perception and translation into control and vehicle responses.

With the most favorable warning conditions, consideration of the analyses shows that spacing maintained between vehicles became independent of speed and indicated that another following system had been established by the driver. Although this system was speed dependent, the drivers employed a different information processing mode and adapted the headways to the remaining constraints on following mostly random variations in speed between two vehicles.

Verification of Predictions Based on Hypothesis

On the basis of the hypothesis for this study, it was predicted that average distance headways between maneuvers would be less with communications than without. Analyses of the data from two-car tests indicated that this was true at the two higher speeds of 45 and 54 m.p.h. but not true for the low speed of 36 m.p.h.

The prediction that the variability in headway would be reduced when the driver of the rear vehicle had prior information about speed changes to be initiated by the driver of the lead vehicle was confirmed by the analyses of test data.

Traffic capacity of freeway lane

Consideration of the data collected during test runs with two cars seems to indicate that a communications system could increase the traffic capacity of a freeway lane. As noted previously (fig. 5), without communications the minimum time headways occurred at speeds below 45 m.p.h. and averaged 2.0 seconds. If the time headways for a single lane of traffic average 2.0 seconds for one hour, the total traffic volume would equal 1,800, which approaches the volume of 2, 000 vehicles per hour that has been widely accepted as the possible capacity for a freeway lane. Few highways in the United States are known to carry a greater volume, and other studies have shown that the minimum headways and maximum capacity usually are obtained when vehicles are traveling at speeds of less than 45 m.p.h.

When communications were added, the speed-headway relationships changed markedly and the mean time headways decreased as the speeds increased. With the 3-second optimum warning time, as illustrated in figure 5, time headways of 1.6 seconds were maintained at 54 m.p.h., the highest speed used in this study. Were time headways to average 1.6 seconds for one hour at 54 m.p.h., the traffic capacity for a single freeway lane would exceed a volume of 2,200 vehicles per hour. Thus theoretically, utilization of a communications system, providing information similar to that of this study, would permit a one-third increase in the traffic capacity of a freeway lane at a speed of 54 m.p.h. This increase was derived from comparison with the study data obtained when the test condition was without communications and the time headway at 54 m.p.h. was 2.3 seconds, which would permit a volume of only 1,600 vehicles per hour.

Therefore, what seems to be a fairly simple communications system would appear, by extrapolation, capable of generating a sizeable increase in the capacity of a freeway at the higher speeds. However, analysis of the three-car data does not allow such a conclusion. The addition of an interposed vehicle eliminated most of the benefits to be obtained from a communications system, as implied by the analysis of data from two-car tests.

As shown in the final column of table 3, without communications the average time headways between the second and third cars were considerably shorter at all three speeds than between the first and second cars with communication of a 3-second warning time. Furthermore, with communication of a 1-second warning and at speeds of 54 m.p.h., the time headways in three-car runs were only 0.1 second less than those maintained during the runs without communications. This elimination of most of the benefits, obtained by use of the communications system, with the interposition of a vehicle in a three-car queue may be attributed to several factors.

First, in a line of traffic a natural tendency exists to maintain stability of speed and spacing. Thus any speed change imposed by the lead vehicle may be expected to be compensated for by the drivers of successive vehicles as they perceive and thereby progressively reduce the effect of the speed change.

Second, if the driver of the third vehicle observed the speed change behavior of the first vehicle directly, he might very well receive as much advance warning as he would from the communications system. That such does take place has been suggested in a report by Forbes (*4*).

Third, automobiles in a line represent a very loosely coupled system, subject to considerable random fluctuations. A communications system such as the one used for this study should have a decreasing effect upon the responses of the driver as the distance increases between the communications system and his vehicle. Any interposed vehicle prevents the following driver from directly coupling himself to the source. The driver, of necessity, must compensate primarily for the behavior of the vehicle immediately preceding his and secondarily for the behavior of those further ahead in the line. In essence, in the multiple car-following situation, the speed change information display represents little more than a cue informing a driver that some change of speed will be made in the preceding line of traffic.

Variability of drivers' responses

Another clear-cut fact obvious from analyses of these data is that a high degree of variability occurs both within and among drivers' responses. Considerable differences were noted in their responses concerning time relations and optimum following, minimum variance conditions, and at different speeds. Although the differences in mean headways with a 3-second warning were quite small at all three test speeds, the analysis of variance showed a significant difference among speeds. As no systematic differences in variance occurred among the three speeds, it seems reasonable to conclude that the significance arises from inter- and intra-driver response changes from run to run. It would appear, then, that the driving system is greatly influenced by the moment-to-moment fluctuations in a driver's psychological state, as well as by his more stable behavioral characteristics. Sensitivity to such subtle qualities is not a characteristic of efficient man-machine systems.

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A Supplementary Study of Concrete Produced by 34-E Dual-Drum Pavers

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This article presents information obtained from tests made during construction of an Arkansas road project on the relation of mixing time to the strength of portland cement concrete produced in a 34-E dual-drum paver. Objective of this supplementary study was to gain data to check against results shown by similar tests made during construction of 16 projects by 13 State highway departments; test results from the previous study were summarized in the April 1960 issue of this magazine.

It was believed that insufficient data had been obtained from the earlier study, especially concerning concrete prepared with short mixing times. Test results from both studies, however, demonstrated that concrete of adequate strength can be obtained from a mixing time as short as 20 seconds, excluding transfer time. Concrete of maximum strength obtained during the supplementary study was produced with a mixing time of 45 seconds. The increased strength ratios shown by tests for various mixing times on concrete from the Arkansas project are believed to have reflected the mechanical efficiency of the mixer used during the supplementary study.

It is noted that more concordant results were obtained from tests made during the previous study. Although no explanation has been determined for some of the varying test results obtained during this study, it is believed that the harshness of the concrete used on the Arkansas project caused some of the exaggeration of the differences in strength exhibited by some of the test results.

Introduction

DURING THE CONSTRUCTION season of 1958, a study of the production of portland cement concrete in 34-E dual-drum pavers was conducted by 13 State highway departments in cooperation with the Bureau of Public Roads. A summary of the findings from that study was published in PUBLIC ROADS in 1960.[2]

When the report on that study was prepared, it was realized that the number of test specimens prepared per variable in each study by the different State highway departments might have been insufficient to average out the uncontrollable variables such as weather and the physical condition of the operators as it might have affected their efficiency. It was believed that insufficient data had been obtained for short mixing times.

The Arkansas State Highway Department had planned to cooperate in the study but delays in the accomplishment of the grading and drainage of the selected project site prevented placement of the pavement in 1958. Subsequently, when the placement of pavement was programed in 1960, the Arkansas Department offered to conduct a supplementary study of the production of portland concrete with the 34-E dual-drum paver and thus provide data that could be checked against the findings obtained from the previous tests.

This study was made during construction of the Arkansas Project F-021-3(8) on a relocation of U.S. Route 67, between North Little Rock and Jacksonville. Construction equipment used included a Koehring 34-E dual-drum mixer built in 1958 and in excellent condition. Equipment in the paving train included a spreader and vibrator, a finisher, a longitudinal float, and a jointing machine. The concrete pavement was covered for 24 hours with wet burlap and then sprayed with a pigmented curing compound. An average of 1,500 feet of pavement, 24 feet wide and 9 inches thick, was placed per 10-hour day.

Concrete design

The concrete was designed in accordance with the Arkansas specifications. The mix contained 5¼ bags of cement per cubic yard and had a maximum water content of 5.5 gallons per bag. The slump was approximately 2 inches, and the air content was between 3 and 5 percent. The aggregates used were natural sand and crushed stone from a local commercial producer. The mix proportions were 94-186-428 by dry weight, which limited sand to only 30 percent of the total aggregate on a solid volume basis. Despite the harsh mix resulting from these mix proportions, a satisfactory finish of the pavement surface was obtained because the concrete was vibrated.

Conclusions

Results obtained from this supplementary study agree with those reported for the previous studies made in 1958, that concrete of adequate strength can be obtained from a mixing time of less than 60 seconds, even with a mixing time as short as 20 seconds. It is possible that the mechanical efficiency of the paver used to mix the concrete for this project permitted the attainment of better results for short mixing times than those obtained in the previous study.

For each mixing time used in this study, a considerable range in compressive strength of the concrete was noted from tests conducted on the groups of specimens collected from different batches of concrete mixture. Also, for each mixing time, the compressive strength of the concrete increased or decreased at reasonably constant rates for certain groups of tested specimens. It is possible that the harshness of the mix was responsible for the extreme strengths noted for some concrete specimens; but, from available data, no valid explanation can be determined for the progressive increase or decrease in strengths noted from tests made on successive batches of concrete.

Program of Study

The preparation of a large number of concrete test specimens for each variable of mixing time was desired for this study. It was believed that more information could be obtained per man-hour by limiting the preparation of specimens to cylinders. Consequently, no beam specimens were prepared for use in this study, and no tests were made for flexural strength of the concrete.

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[1] This article was presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] *A Study of 34-E Dual-Drum Pavers*, by D. O. Woolf, PUBLIC ROADS, vol. 31, No. 1, Apr. 1960, pp. 1-10.

The planned program included requirements for the following listed operations: Tests to be made on concrete prepared with mixing times of 20, 30, 45, and 60 seconds, exclusive of transfer time; use of only one overload, of 20 percent; preparation of 108 specimen cylinders from concrete of each mixing time; and completion of a suitable number of tests for consistency, air content, and unit weight. Because of the excessive rainfall that interrupted the study period (June 21–July 21, 1960), it was necessary to deviate from the planned program and only 72 specimens were prepared from concrete of some mixing times. However, all the specimens prepared for each batch were divided into two equal groups; half were used for tests made at 7 days of age and the other half were used for tests made at 28 days.

Sampling procedure

Test samples were taken from the last batch of concrete placed on the subgrade before the paver backed to place the top course. Batches were sampled every 20 to 30 minutes. These samples, approximately 2 to 3 cubic feet of concrete, were shoveled into a pan from five locations in the pile. The pan was carried to a truck, which took the pan to the sample preparation site established for each day's work.

It had been planned to test every sample for consistency by the Kelly ball penetration and slump measurement tests and for air content by the Chace meter; then six 6- by 12-inch cylinders were to be cast. The Kelly ball tests were made on the concrete, which had been placed on the subgrade from the test bucket load, and the other tests were made at the sample preparation site. Tests for unit weight and determinations of air content by a pressure meter were scheduled to be made on alternate batches. The concrete cylinders were cast in cardboard molds with metal bottoms, and the completed specimen cylinders were covered with 5 layers of wet burlap. On the following morning, the molds were stripped, the cylinders were marked and then stored in tanks of water. The cylinders were taken to the Arkansas State testing laboratory one day before the date of testing. They were capped with a sulfur compound for the compression tests.

Mixing procedure

Personnel from the Bureau of Public Roads timed the mixing for the test batches of concrete. The contractor usually used a mixing time of 50 seconds. Because of poor subgrade conditions outside the forms, however, the paver could not operate continuously at a fixed mixing time as the batch trucks could not always back to the skip, discharge, and clear the skip in the required period of time. Consequently, to assure the validity of the study, when samples were to be taken four consecutive batches were mixed for the specified length of time, and the samples were taken from the fourth batch. To obtain samples from concrete mixed for 20 seconds,

Table 1.—Summary of Arkansas project concrete production test data

Samples taken	Air temperature	Consistency		Air content		Unit weight	Average compressive strength at –	
		Kelly ball	Slump cone	Chace meter	Pressure meter		7 days	28 days
60-SECOND MIXING TIME								
June 21, 1960:	°F.	In.	In.	Pct.	Pct.	Lb./c.f.	P.s.i.	P.s.i.
2:45 p.m.			4.0	6.0			2,620	3,320
3:15 p.m.		4.0	3.7	6.9		141	2,610	3,180
4:00 p.m.		1.5	2.5	4.7	3.8		4,040	4,940
4:40 p.m.		1.2	1.6	4.3		148	4,110	4,990
July 20, 1960:								
9:00 a.m.	88	2.0	2.5	5.2	3.8		3,440	4,150
9:25 a.m.	94	1.5	2.4	5.0		148	3,220	4,210
9:45 a.m.	94	1.8	1.6	4.0	3.9		3,590	4,370
10:15 a.m.	91	2.0	1.9	5.0		146	3,400	4,260
10:35 a.m.	98	1.4	1.5	5.0	4.0		3,730	4,760
11:00 a.m.	98	1.3	1.4	5.0			3,610	4,600
11:20 a.m.	92	1.0	1.9	4.0		150	5,190	6,230
11:40 a.m.	89	1.2	2.2	4.0	3.8		4,210	4,730
Averages		1.7	2.3	5.0	3.9	147	3,650	4,480
45-SECOND MIXING TIME								
June 22, 1960:								
10:10 a.m.	98	2.0		4.3	4.3		3,680	4,500
10:55 a.m.	98	2.8	2.6	5.2		144	3,190	3,950
11:30 a.m.	98	1.8	0.8	3.9			3,150	3,980
11:55 a.m.	99	2.8	3.8	6.0	4.9		2,770	3,800
1:50 p.m.		2.8	1.3	4.3			4,000	4,660
2:20 p.m.	99	2.0	1.5	5.6		146	3,410	4,420
2:45 p.m.	98	1.1	1.5	3.5	3.8		4,370	5,510
3:15 p.m.	98	1.9	0.3	4.7		146	3,420	4,100
3:50 p.m.		1.8	1.1	4.3			3,170	4,180
July 20, 1960:								
1:20 p.m.	85	1.5	2.5	4.7	5.2		3,840	4,700
1:45 p.m.	85	0.6	1.0			149	3,810	5,780
2:00 p.m.	83	1.0	0.9	3.5	2.7		3,930	5,410
2:20 p.m.	83	1.5	1.9			147	3,930	4,820
2:35 p.m.	82	2.0	2.6	2.7	3.2		3,560	4,660
2:50 p.m.	83	2.2	2.4			148	2,940	4,140
3:10 p.m.	85	0.7	0.8			150	4,730	6,060
3:30 p.m.	83	2.0	3.0		4.2		2,720	3,470
Averages		1.8	1.8	4.4	4.0	147	3,570	4,600
30-SECOND MIXING TIME								
June 23, 1960:								
10:35 a.m.		1.6	2.5	4.7	5.1		3,150	4,170
11:00 a.m.		1.6	1.2	4.3		145	3,150	4,200
11:35 a.m.		0.9	0.9	4.3			4,270	5,680
11:55 a.m.		1.3	1.7	4.3	3.9		4,280	4,450
1:35 p.m.		1.4	2.4	4.3		146	3,380	4,160
2:10 p.m.		2.2	2.5	4.3			3,480	4,600
2:45 p.m.	95	0.9	1.8	4.3	3.6	146	3,120	4,240
3:15 p.m.	96	2.0	2.5	4.3			2,620	3,440
3:35 p.m.	95	2.4	2.0	4.3			2,670	3,830
July 21, 1960:								
9:30 a.m.	92	1.2	2.0	6.4	4.8		3,220	3,800
9:55 a.m.	95	1.8	1.6	6.4		148	3,180	3,960
10:20 a.m.	94	1.4	1.5	6.0	3.3		3,960	4,410
10:55 a.m.	96	0.5	0.9	4.3			4,260	5,120
11:10 a.m.	86	1.2	0.6	4.7	3.1		4,480	5,600
11:40 a.m.	91	2.3	1.9	5.2		145	3,540	4,170
Averages		1.5	1.7	4.7	4.0	146	3,520	4,390
20-SECOND MIXING TIME								
June 24, 1960:								
9:45 a.m.	90	1.7	2.5	4.7	4.6		2,840	3,700
10:15 a.m.	90	2.4	2.5	3.5		146	2,990	4,160
10:40 a.m.	91	1.6	2.2	4.3			2,690	3,590
11:00 a.m.	86	1.6	2.2	3.9	4.4		2,470	3,290
11:25 a.m.	89	0.5	0.6	3.9		147	5,070	[1] 6,220
11:45 a.m.	78	2.0	3.4	4.3	4.3		2,760	3,610
July 21, 1960:								
1:25 p.m.	89	1.5	2.6	4.3	3.7		2,490	3,440
1:45 p.m.	86	1.5	2.0	5.6	3.8		2,710	3,250
2:10 p.m.	84	1.3	2.1	5.3		147	3,600	[1] 4,900
2:25 p.m.	81	1.0	1.7	3.9		148	4,280	4,360
2:40 p.m.	80	1.4	2.5	3.9	3.8		2,300	3,540
2:55 p.m.	78	2.2	1.5	3.1		149	4,160	[1] 4,900
Averages		1.6	2.2	4.2	4.0	147	3,200	[1] 4,100

[1] These averages include the "wild" values for individual specimens.

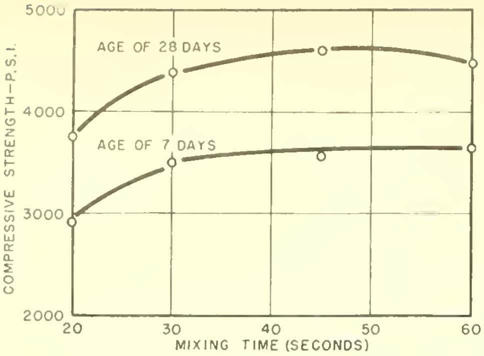

Figure 1.—Effect of mixing on compressive strength of concrete.

it was necessary to operate the paver manually and only one batch of concrete was mixed at a time. No appreciable difference in the appearance was observed for the concretes mixed for different lengths of time; the aggregate seemed to be well coated even in the concrete mixed for the shortest length of time, 20 seconds.

When this study had been about half completed, heavy rains and the resultant soil conditions halted construction on the project for a period of about three weeks. The study was subsequently completed without incident.

Although not pertinent to this study of concrete, it is of interest to note the plan devised by the contractor in an attempt to resume construction as soon as the weather had cleared. After the rain, the soil condition of the grade beside the forms was such that trucks could not be operated on it. To overcome this, the contractor tried double batching; that is, using two pavers, one on the service road and one on the grade. After a batch had been mixed dry in the first paver, it was dumped into the skip of the paver on the grade and, with the addition of water, mixed to form concrete. This procedure was conducted for only a few hours when the inspector closed the project because of spillage of materials.

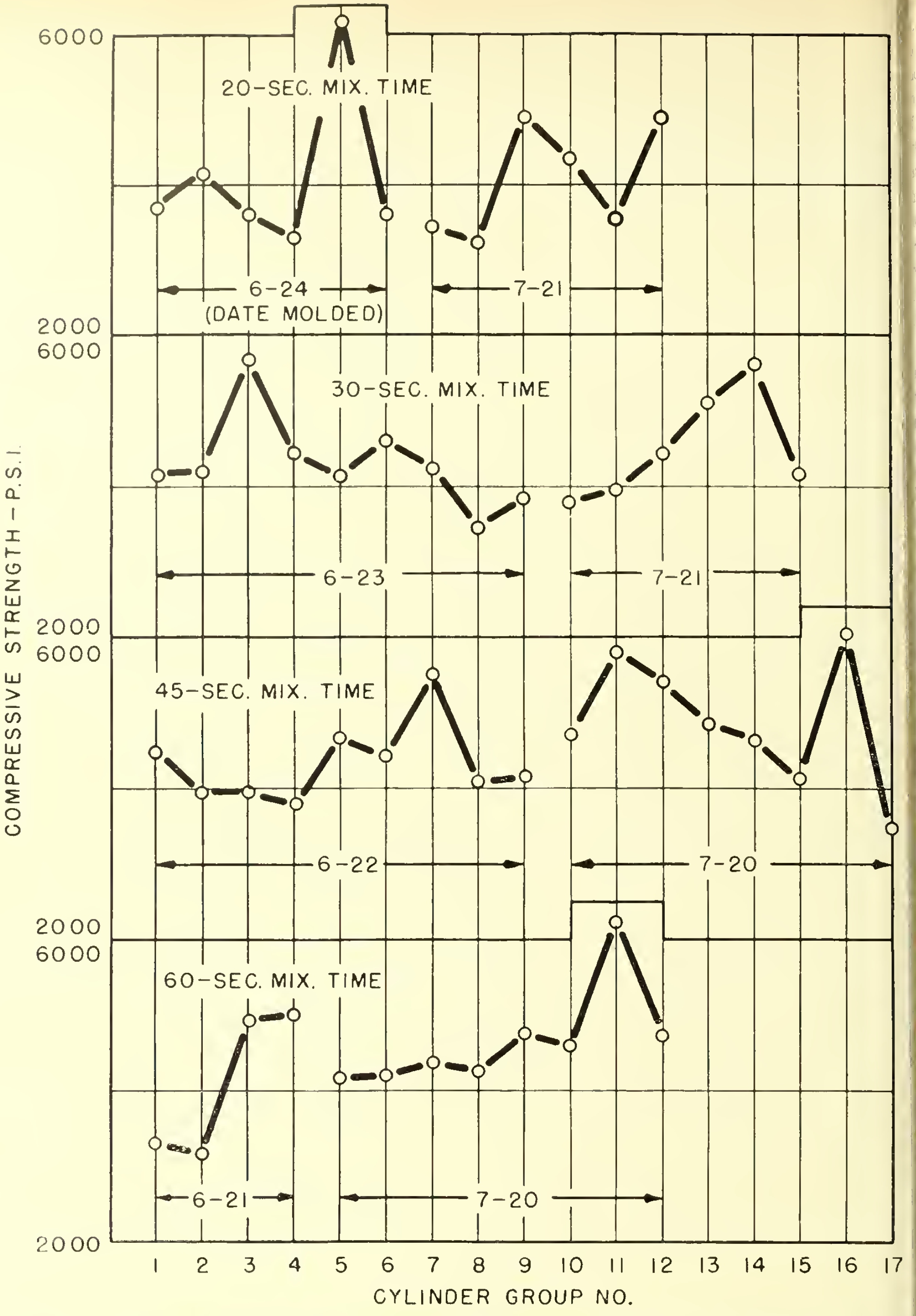

Figure 2.—Compressive strength of concrete at 28 days; groups of test cylinders arranged in chronological order of molding.

Effect of Mixing Time

Six cylinders were prepared from each batch of concrete tested. Compressive strength tests for the concrete were made on half of these cylinders at an age of 7 days and on the other half at an age of 28 days. Average values for the compressive strength determined for each group of three cylinders are given in table 1. Data are also shown on consistency, air content, and unit weight of the concrete.

The relation between mixing time and strength of the concrete is shown by the curves in figure 1.[3] Of the two curves, greater weight is given to the one representing results from tests made on specimens 28 days old. These curves show that the maximum strength of concrete tested in this study was obtained with a mixing time of 45 seconds. (All the mixing times shown exclude the transfer time.) With the results for the 60-second mixing time considered as unity, the strength ratios of the concrete for the various mixing times were:

Mixing time (seconds)	*Strength ratio (percent)*
60	100
45	103
30	98
20	[3] 84

The results shown for mixing times of less than 60 seconds are somewhat superior to those obtained in the preceding study. However, results from the tests of both studies demonstrate conclusively that mixing times as short as 30 seconds could be used with little reduction in compressive strength of concrete. The small differences shown by the two sets of test results probably reflect a difference in the mechanical efficiency of the mixers used.

Variations in Test Results

The compressive strengths of the concrete at 28 days, obtained from tests made on each group of cylinders and arrayed chronologically, are shown in figure 2. Attention is called to some possibly unusual trends exhibited by the concrete specimens molded consecutively on the same day. For example, at 45 seconds of mixing time the specimens in groups 11–17, with one exception, had an almost uniform rate of loss of strength. The exception, the three specimens of group 16, had the highest compressive strength of any group of specimens tested in this series. However, specimens from group 17, the last of this series, had a low strength value which conformed to the general trend shown by groups 11–15.

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[3] For the 20-second mixing time, compressive strengths of six cylinders were omitted because they were so unusually high as to appear questionable.

For the other mixing times, similar progressive increases or decreases of compressive strength of the concrete specimens are shown in figure 2. For a mixing time of 20 seconds, two sets of three groups of specimens (2–4 and 9–11) had uniform decreases in strength. For a mixing time of 30 seconds, five groups of specimens (10–14) had a fairly uniform rate of increase in strength. For a mixing time of 60 seconds, no corresponding marked changes in strength were noted, although groups 5–12 had a slight but progressive increase in compressive strength with the exception of the specimens from group 11. The "wild" test results obtained from group 11 were similar to those obtained for certain single groups of specimens at each of the other mixing times, such as was noted for group 16 at 45-second mixing time.

Concrete strength test results

No definite relationship for all of the cases of progressive increase or decrease in compressive strength of the specimens tested at an age of 28 days can be shown by the data collected during this study. As will be discussed later, available data have been correlated with the results noted from tests of one of these cases. But for the other cases, some assumptions (which could be called guesses), have been made.

Most of the test results showing progressive increases or decreases in compressive strength of the concrete were obtained from specimens made in July 1960. All but one group of the specimens from the 60-second mixing time, which were molded during the morning of July 20, had increasing strengths. Specimens from the concrete with a 45-second mixing time were molded during the afternoon of July 20, and six of the eight groups of cylinders had progressively decreasing strengths.

The concrete specimens molded on the following day, July 21, showed a pattern of behavior similar to that obtained in tests on the specimens from the previous day's molding. The specimens molded in the morning were from the concrete with a 30-second mixing time and, with one exception, had progressively increasing strengths. Specimens from the concrete with a 20-second mixing time were molded in the afternoon and had varying strengths. Of the six groups of cylinders tested, specimens from three consecutive groups had steadily decreasing strengths, those from the first two groups of the series had lower strength values, and those from the last group had higher strength values.

Explanation of Unusual Test Results

To explain unusual results obtained from tests made on concrete during construction operations, writers frequently refer to items for which no determined values are available. The temptation to do that in this article is quite strong. Of all the various items that would have had a marked effect on the strength of the concrete but for which data are not available, the first choice is the water content of the fine aggregate. Normal variations could have been expected in this water content and might have caused the unusual test results. The receipt and use of a new lot of sand in a moist condition could have caused the strength of the concrete to decrease. Use of sand stockpiled and subjected to the high atmospheric temperatures that prevailed during much of the time of this study could have caused the strength of the concrete to increase. However, in either case any surplus or deficiency of water in the sand used should have been reflected in changes in the consistency of the concrete.

Tests on Concrete Batches

Figures 3 and 4 show the average values obtained from various tests performed on batches of concrete mixed for 20, 30, 45, and 60 seconds, respectively. For each mixing time, test data are presented that show average compressive strength, penetration determined by the Kelly ball method, slump measurement, and air content determined by the Chace meter. In several cases, air content data were not available.

Concrete, 20-second mixing time

The average test values obtained for concrete batches from the 20-second mixing time, shown in figure 3, do not provide a clue to the reason for the two series of progressively decreasing strengths. On the contrary, the test data obtained for consistency and air content of the concrete are in opposition to the trend of the results obtained from strength tests. Little basis for an explanation of the test performance of concrete from batches 2–4 and 9–11 is provided by these data. The high strength shown for batch 5 is reflected, however, by the decrease in consistency measured by the Kelly ball penetration and slump tests.

Concrete, 30-second mixing time

Information concerning the concrete produced with a 30-second mixing time is shown in figure 3. Tests showed the concrete from batches 3 and 14 to have high compressive strength and that from batch 8 to have the lowest strength of the 15 batches tested. Tests on concrete from batches 10–14 showed a set of progressively increasing strengths.

The records from slump tests of the concrete with a 30-second mixing time for batches 10–14 show a steady decrease in consistency, which could well be associated with the increase in strength recorded for these same concretes. The Kelly ball test results for these batches of concrete are irregular, as are the determinations for air content. It seems to be somewhat questionable that a change in slump from 2.0 inches to 0.6 inch could cause a change in compressive strength from 3,800 p.s.i. to 5,600 p.s.i. Possibly, it would be proper to consider these slump measurements as indications that a change in consistency occurred, which must have been accompanied by some other change that affected the compressive strength of the concrete and caused such a variation.

The high strengths shown by the tests on concrete from batches 3 and 14, and the low strength shown for concrete from batch 8, are associated with the low and high slump measurements, respectively. The results of the Kelly ball penetration tests showed the same degree of association with the strength of the concrete from various batches as those for the slump tests, but were not of similar magnitude.

Concrete, 45-second mixing time

Test data recorded in figure 4, for the concrete prepared with a 45-second mixing time, show that three batches of concrete had high or reasonably high strength values whereas one had a low value. With the exception of the highest strength value recorded, for batch 16, a progressive decrease in strength was noted for a series of seven batches of concrete (11–17). The first batch of this series, number 11, had a strength of 5,780 p.s.i. at 28 days and, except for batch 16, the strengths of the concretes of this series decreased almost uniformly to the 3,460 p.s.i. recorded for batch 17.

A progressive increase in penetration (a decrease in consistency) was shown by Kelly ball tests for the concrete from batches 11–15. However, the concrete of the 16th batch had only a small penetration value and that of the 17th batch was only slightly above the average for concrete in this series of batches.

The slump measurement for batch 17 was of some magnitude, but the slump measurements for batches 11–16 followed no definite pattern.

An insufficient number of Chace meter tests was made for the air content determinations of concrete in this series of batches to be of any assistance in explaining the progressive decrease in strength for this series.

Concrete, 60-second mixing time

Data from the tests on concrete prepared with a 60-second mixing time, as seen in figure 4, showed only a few correlations between strength and consistency or air content of the concrete from various batches. Concrete in batches 1 and 2 had relatively low values for strength and high values for consistency. Batch 2 concrete also had a high air content recorded by the Chace meter test. Batch 11 concrete had the highest strength, but the determinations for consistency or air content failed to show any reason for this. Concrete batches 5–10 in general showed an irregular but small increase in strength from 4,150 p.s.i. to 4,600 p.s.i. Similarly, a small decrease in slump measurement was noted for these same batches, but the Kelly ball and Chace air meter tests showed no trends in results similar to those obtained in tests for strength.

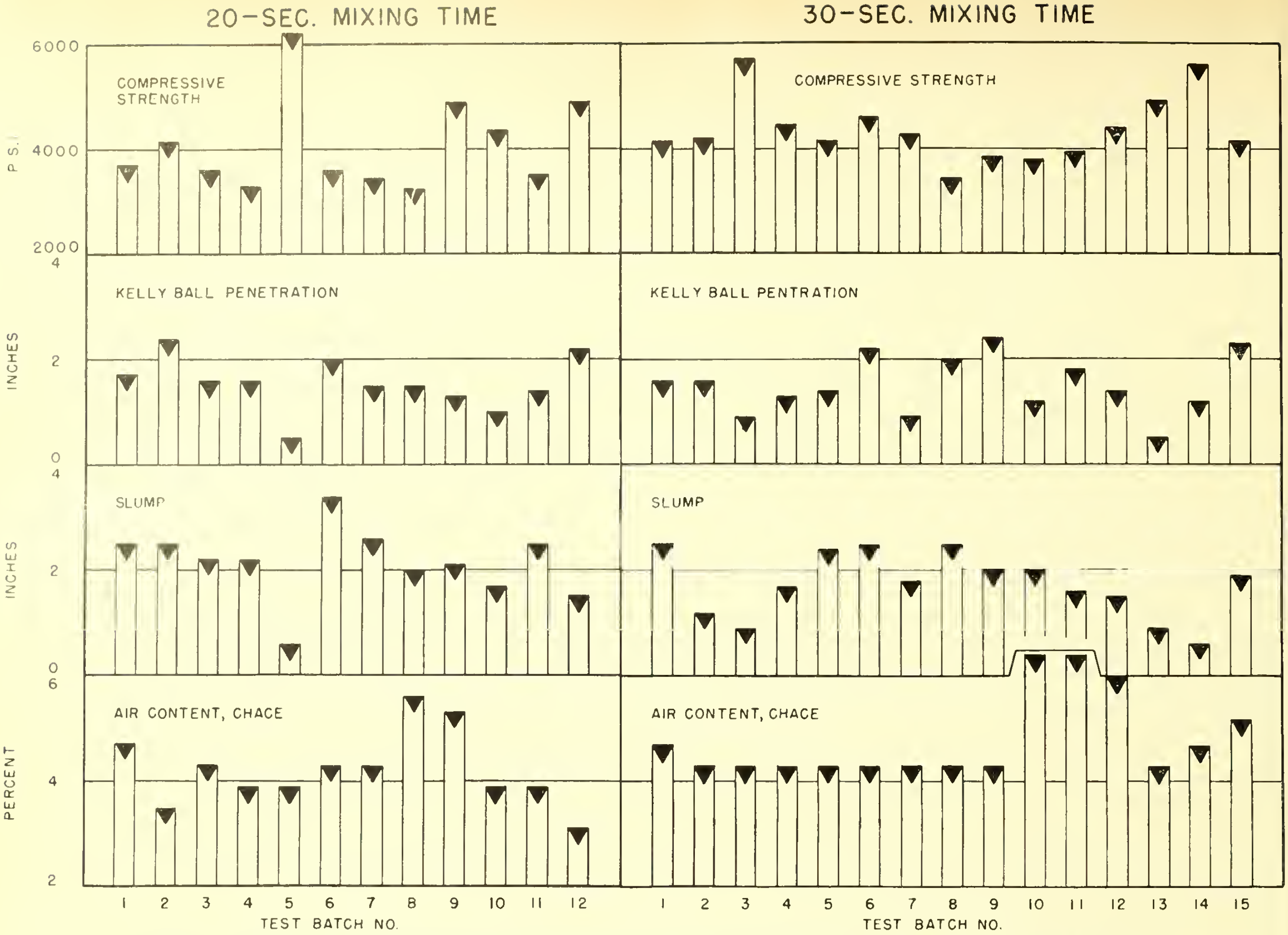

Figure 3.—Average values determined from various tests on batches of concrete mixed for 20 and 30 seconds.

Correlation of Test Results

In three out of four cases, more definite correlation was found between the test results for compressive strength of concrete and the slump measurement than between the test results for strength and either the Kelly ball penetration or the Chace air meter determinations. This was somewhat of a disappointment. It had been hoped that the Kelly ball and Chace meter test results would correlate closely with results of the strength tests of concrete. Because these tests for consistency and air content can be made quickly, it had been hoped that they could be established for use as acceptance tests with the definite knowledge of the close relationship of such test results with the strength of concrete. Such a relationship, however, was not established by test results of this study and the slump test remains the more reliable indication of the quality of concrete.

Chace Air Meter Test Results

Concretes of this study subjected to tests made with the Chace meter had an average air content value of 4.6 percent. This result is sufficiently different from the average air content value of 4.0 percent obtained from the tests made with the pressure meter to warrant caution in accepting test results obtained with the Chace meter. An air content of 4.0 percent is believed by many authorities to be about the lowest percentage of air that can be permitted and at the same time assure a concrete having adequate resistance to the effects of freezing and thawing. Increase in the air content to 4.6 percent should be accompanied by a marked increase in the durability of the concrete. Consequently, the results obtained in this study with the Chace meter indicate a durability for the concrete that may be misleading. As mentioned by others in reports on studies of the Chace meter, the test results obtained with this instrument should be considered to indicate general ranges in air content, i.e., high, medium, or low, and more precise indications should not be expected.

Control of Concrete Strength

In his paper on the probabilities of obtaining concrete of uniform strength, *How Good Is Good Enough*, presented at the February 1961 Convention of the American Concrete Institute,[4] Edward A. Abdun-Nur makes reference to the excellent Bureau of Reclamation control of the strength of concrete. Under this control, a coefficient of variation of practically 15 percent is obtained. In the analysis of the data for the Arkansas project, it was decided to determine the coefficient of variation for each group of data and to compare the values thus obtained with those for the mean deviation from the average. This latter step was taken because the mean deviation is considered less difficult to compute and is more readily understood. In addition, frequency distribution curves were prepared from the 28-day test results for the specimens representing each of the four mixing times.

The frequency distribution curves are shown in figure 5. The curve for the 20-second mixing time indicates that most values lie in a range of from 3,100 p.s.i. to 4,300 p.s.i. There were, however, some quite high test results, which caused a marked misshape of the curve and a shift upward in the average value. In the interests of reliability of the findings of these tests, it might be appropriate to classify the six highest test results for single cylinders as sufficiently "wild" to warrant

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[4] Journal of American Concrete Institute, vol. 59, No. 1, Jan. 1962, pp. 31–45.

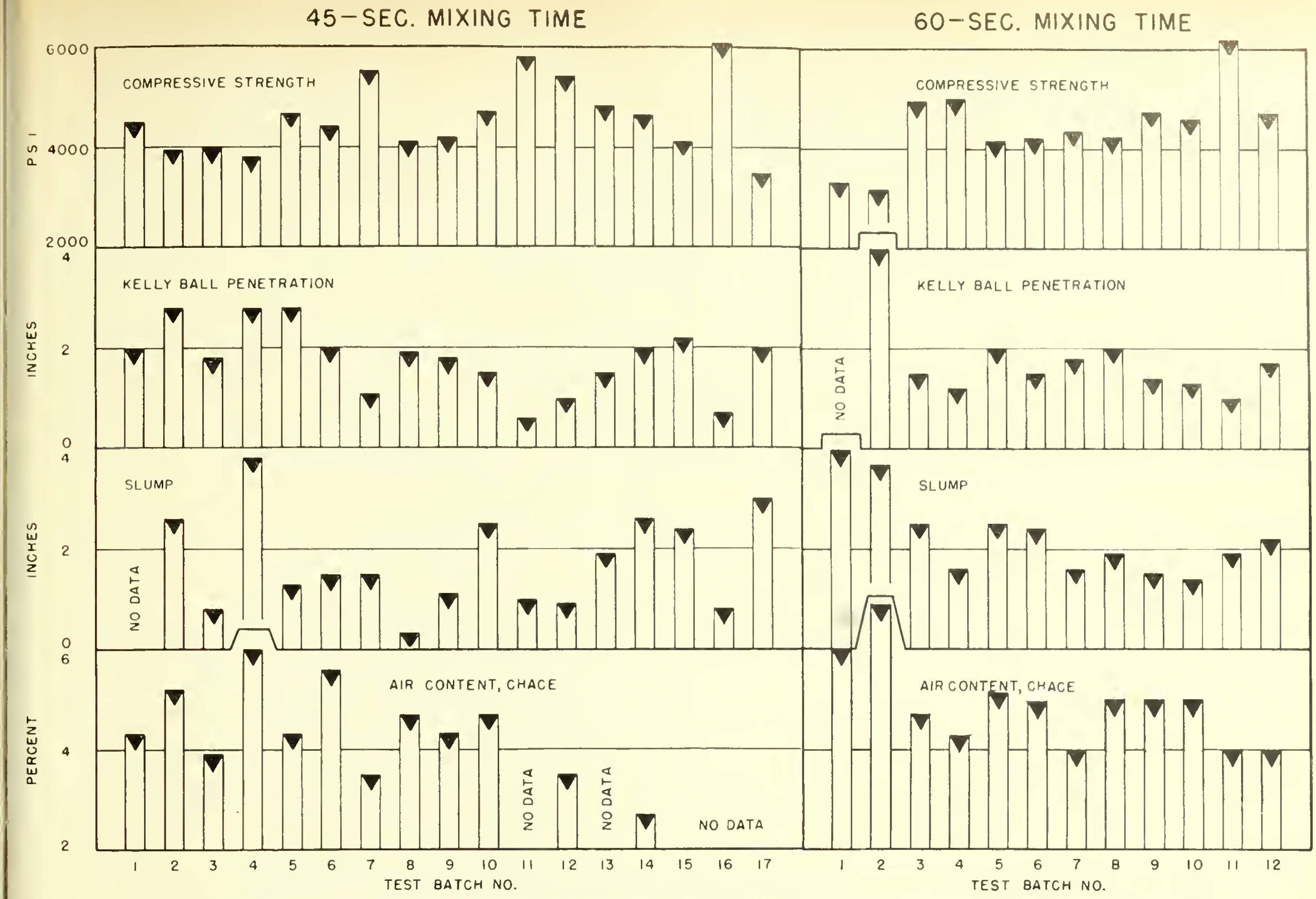

Figure 4.—Average values determined from various tests on batches of concrete mixed for 45 and 60 seconds.

their rejection from the data considered. If this were done, the average value for the remaining test values would become 3,750 p.s.i., a value more in keeping with the other findings of the study.

Although the curve for test data for concrete from the 45-second mixing time also shows a lopsidedness or skewness (as did that for the 20-second mixing time), the test data fail to show any particular point at which higher values might be considered "wild" or unreliable. Consequently, no adjustment of the average value has been attempted for these test data.

Values for coefficient of variation and mean deviation from the average are plotted in figure 6. The values determined by tests on concrete from the 20-second mixing time do not include the six "wild" results previously mentioned. Had these been included, the coefficient of variation would have been 21.8 percent, and the mean deviation from the average would have been 17.2 percent. For the values presented, it is interesting to note that the coefficients of variation are close to the 15 percent mentioned by Abdun-Nur as denoting excellent control of concrete strength.

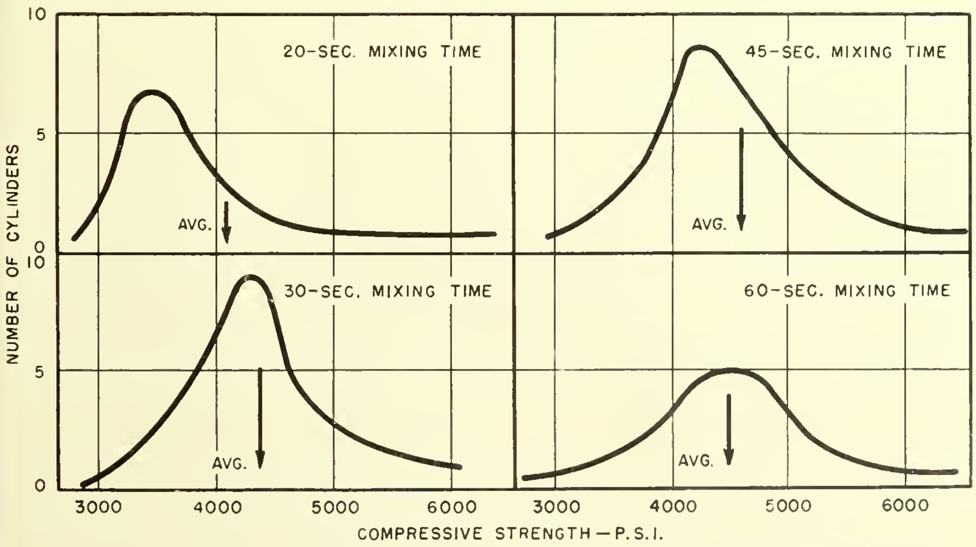

Figure 5.—Frequency distribution of compressive strengths of concrete at 28 days.

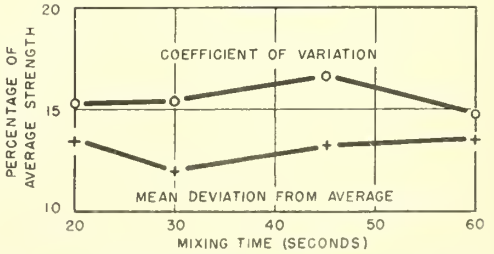

Figure 6.—Comparison expressions for uniformity of compressive strengths of concrete.

It is also noted that the mean deviation from the average follows closely the trend of the coefficient of variation.

On the basis of the frequency distribution curves shown in figure 5, the most concordant test results are shown to have been obtained with concrete mixed for 30 seconds. Some concern has been expressed about the nonuniform test results obtained during this study. As shown in figure 5, values markedly different from the average were obtained for some specimens prepared from concrete of each mixing time. The test data associated with the strengths of the concretes do not indicate why such variable results were obtained.

From a review of the data obtained in the 1958 studies for the different mixing times of the concrete, it was noted that the test results had generally been more concordant. Also, it was observed that when the concrete tested during the previous study had been described as harsh, there had been a tendency for a marked range in strength test results. It is believed that the harshness of the concrete used on the Arkansas project caused some exaggeration of the differences in strength normally found in tests of concrete made during construction. With a more plastic concrete, even lower values for coefficient of variation should be obtained than those reported here.

Arkansas Tests

Table 2.—Compressive strength of pavement determined from test cores

Mixing time	Number of specimens	Average compressive strength
Seconds		*P.s.i.*
20	2	3, 760
30	3	4, 180
45	4	4, 060
60	3	4, 530

In accordance with the requirements of their standard specifications, the Arkansas Highway Department drilled cores of the pavement for use in making determinations of the thickness of the slab and the compressive strength of the concrete. The compression tests were made on cores taken from the pavement at an age of 3 months. Results from tests on 59 cores showed the concrete to have an average strength of 4,430 p.s.i. with maximum and minimum strength values of 6,060 p.s.i. and 2,410 p.s.i., respectively. Strengths for the experimental sections of the pavement, determined from tests on the cores, are shown in table 2. These values do not agree with those determined in tests of the cylinders, but they do show that concrete of adequate strength was furnished even when mixing time was limited to 20 seconds.

Errata

In the December 1961 issue of PUBLIC ROADS, vol. 31, No. 11, a printer's error appears on page 222, in table 1, of the article, *A Laboratory-Field Study of Hot Asphaltic Concrete Wearing Course Mixtures.* The following words, which appear in the eighth and ninth lines of the first column of table 1: "has confusing appearance. Set [2]—" should be deleted.

Absolute Viscosity as a Specification Control for Bituminous Binders

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by J. YORK WELBORN, Chief,
Bituminous and Chemical Branch,
and WOODROW J. HALSTEAD, Head,
Bituminous Materials and Chemical Section

The potential application of absolute viscosity as the basis for controlling the consistency of bituminous materials is of great interest to those concerned with the procurement and use of such materials. The historical development of present specifications, and the advantages to be gained by a change that would base specification requirements on kinematic values, are presented in this article. Problems related to the adoption of absolute viscosity as the control for both asphalt cements and liquid asphalts are pointed out—more complex problems are noted for asphalt cements.

A brief description is included of some of the instruments now available for determination of viscosity of each type of asphaltic material. Included in a related article appearing elsewhere in this magazine is a comparison of the Zeitfuchs cross-arm viscometer, for the determination of kinematic viscosity; and the Saybolt furol viscometer, now used for determination of furol viscosity.

Introduction

WHEN ASPHALT cement first came into use in the United States as a paving material, it was common practice for the engineer or chemist to determine its hardness by chewing a small portion of the material. It soon became evident to these pioneers that consistency was a very important property of bituminous material and, during the period between 1890 and 1910, the penetration test and other tests were devised to obtain more precise information about the relative consistency of bituminous materials.

Technology in nearly all fields of science and engineering has advanced at a tremendous pace since 1900, and the knowledge of highway materials possessed by engineers today is much greater than that of the pioneers. The highway engineer now uses consistency values to select a bituminous binder for a particular use and also as a basis for adjusting certain construction operations. The researcher has established the importance of consistency on the ultimate performance and durability of pavements. Despite such recognition of the importance of consistency of bituminous material, most of the standard tests being used today to measure this property are essentially the same as those developed by the pioneers.

Tests *currently in use*

There is little doubt that the tests now in use have done and are doing a creditable job; however, they have an inherent weakness in that they are empirical tests and measure consistency in a wide variety of units, which have only an indirect relation to viscosity. For example, the original consistency of liquid asphalt products is measured by the Saybolt furol viscometer in seconds, a unit of time. After distillation, the consistency of the residues of slow-curing materials is measured by the float test in units of time. The residues of the medium- and rapid-curing cutbacks are measured by penetration in units of length. The measurements of the Engler viscometer, used for testing consistency of tar products, are reported as the relation of the product's consistency to the viscosity of water; and the results of the softening point test used for both tar and semiasphaltic materials are reported in terms of temperature. Results obtained from all of these tests depend on the type of instrument used and the conditions of the test.

Because of the variety of tests and units of measurement, the highway and research engineers are hampered in relating data from one type of test with that obtained from another. Often, fundamental relationships involving viscosity cannot be determined because of the failure of the empirical consistency test to measure the viscosity accurately. Thus, the need for tests that will measure the consistency of all types of asphaltic or tar road materials in one basic unit is quite evident, and the logical choice for such a unit is the fundamental unit of viscosity.

While this idea is not new, having been suggested from time to time by a number of different writers almost from the very beginning of bituminous highway construction in the early 1900's, the move at the present time has more impetus and the backing of a greater segment of both research and testing organizations. The Highway Research Board's special committee on problems of mutual interest to consumers and producers of asphaltic materials has urged that studies be conducted and a concentrated effort be made to develop test methods and apparatus so that specifications can be based on viscosity in absolute units. The AASHO committee on materials is considering the problem, and research studies are being conducted by a number of State highway departments, the Asphalt Institute, individual producers of asphalt, and the Bureau of Public Roads.

Although the change to absolute viscosity as a specification control is clearly desirable from an academic standpoint, the fact that the empirical tests are an integral part of the highway engineer's thinking cannot be overlooked. Engineers understand the significance of results of the present tests and their differences in values; thus, it is important that the effect of any changes to absolute viscosity as the standard unit of measurement for asphalt consistency be considered carefully. The purpose of this article is twofold; namely, to consider in a general way the primary problems connected with specification control by means of absolute viscosity for both liquid asphaltic products and asphalt cements, and to review briefly the instrumentation that is available for determining viscosity for each type of material.

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[1] This article was presented at the 47th annual meeting of the American Association of State Highway Officials, Denver, Colo., Oct. 10, 1961.

Brief Review of Basic Concepts

A brief review of the basic concepts involved and the definition of absolute viscosity are presented to point out some of the fundamental difficulties that must be overcome before specification control of consistency in terms of absolute viscosity units can be adopted for all types of bituminous materials.

Bituminous materials, being essentially viscous liquids, are extremely susceptible to viscosity changes as temperatures change. When expressed in absolute units, the viscosity of an asphalt cement at 32° F. is a billion times its viscosity at the mixing temperature (275° to 300° F.) of pavement materials. At the present time, no single instrument is available to measure viscosity over such a wide range.

The research technologist needs to define correctly the rheological properties (flow) of the bituminous material; to determine the manner in which these properties change with changes in temperature; to establish the relationship of the rheology of the bituminous binder to the characteristics and behavior shown by the paving mixture in the laboratory, during construction, and on the highway; and to prepare specifications that will control the important factors within the desired ranges. To produce deformation and flow, a force is necessary. Thus, in engineering terms, the primary interest of the technologist is in the stress/strain relationship of the viscous material.

When the stress/strain relationship is such that the strain (flow) is directly proportional to the stress, the material is a true (Newtonian) liquid. When the shearing stress of a Newtonian liquid is plotted against the flow produced by that stress, a straight line through the origin results. The numerical value of the slope of the line, when expressed in the proper units, is the coefficient of viscosity of the material, simply referred to as the viscosity. However, straight lines are not obtained for many viscous materials when the shearing stress is plotted against the rate of shear. The curve may bend upward, downward, or fail to pass through the origin. Various rheological terms are used to describe specific types of deviation from the straight line, but in this article such deviations are referred to as complex-flow or non-Newtonian properties of the bituminous material. Most asphalt cements will exhibit such complex-flow properties to some extent at atmospheric temperatures. The degree of complexity will vary for different materials.

Absolute viscosity

The unit of absolute viscosity is the poise. When a shearing stress of one dyne per square centimeter induces a unit rate of shear, the coefficient of viscosity is one poise. The unit rate of shear is obtained when two parallel planes of the liquid, one centimeter apart, have a velocity of one centimeter per second with relation to each other. The liquid in between the reference planes is considered to consist of innumerable parallel planes, each of which has a relative velocity that is proportional to its distance from the reference planes. It should be noted that the fundamental unit of the rate of shear in this usage is not centimeters per second as might be implied by the terminology used but is centimeters per second per centimeter, which reduces to 1/sec. This is usually referred to as seconds $^{-1}$ or reciprocal seconds.

Kinematic viscosity

Kinematic viscosity is often used instead of absolute viscosity, and its unit of measurement is stokes. This unit is the result obtained by dividing the absolute viscosity in poises by the density of the liquid under test. Kinematic viscosity is very useful for laboratory testing because it can be obtained directly without knowledge of the density of the liquid by measuring the flow of the material in a capillary tube. Measurements in both kinematic viscosity in stokes and absolute viscosity in poises, have been used in studies made with bituminous road materials. For asphalt cement, there is essentially no difference in the numerical value of viscosity, whether expressed in terms of poises or stokes, because the density for this material is close to 1 gram per cubic centimeter. In reporting absolute or kinematic viscosity, for convenience, the measurements are sometimes expressed in terms of centipoises or centistokes, which are one-hundredth of a poise or stoke, respectively. Also, the terms megapoise or megastoke are used to denote one million poises or stokes, respectively.

The problems relating to the application of fundamental units of measurement for absolute viscosity to the liquid asphalts are quite different from those relating to asphalt cements of the penetration grades; therefore, each type of material will be discussed separately.

The Problems With Respect to Liquid Asphalts

A survey of State specifications by the Bureau of Public Roads, in 1931, revealed a great difference of opinion as to what viscosity limits, types of viscometers, and test temperatures should be used to define the consistency of liquid asphaltic road materials being used at that time. This had resulted in numerous specifications and often had caused confusion in the production, control, and utilization of the products.

During 1932, the Asphalt Institute and the Bureau of Public Roads promoted and held a series of conferences with representatives of the State highway departments in an effort to simplify and standardize specifications for the rapid (RC), medium (MC), and slow (SC) curing asphaltic road materials. The outcome was the development of specifications for four grades of rapid curing and six grades of medium curing cutbacks, which were adopted by the Asphalt Institute in 1932, by the Federal Specification Board in 1933, and with minor revisions by the AASHO in 1938. Specifications for five grades of SC materials were also recommended and have been used by some agencies, but these have not been adopted by the AASHO.

These specifications represented a great improvement over the previous specifications by providing standards for systematic grading; however, uniformity was still lacking in the viscosity ranges specified for the various grades of different types of materials. Four temperatures: 77° F., 122° F., 140° F., and 180° F. were used when measuring the furol viscosity. It was found that when the viscosity limits were converted to one temperature, 140° F., the same number or grade designation did not indicate the same viscosity limits for SC, MC, and RC products, and the total range of viscosity was different for each type of material. Also, the gaps between consecutive grades were variable and, in two cases, the maximum viscosity of one grade of material was higher than the minimum viscosity of the material of the next heavier grade. These relations are shown in figure 1.

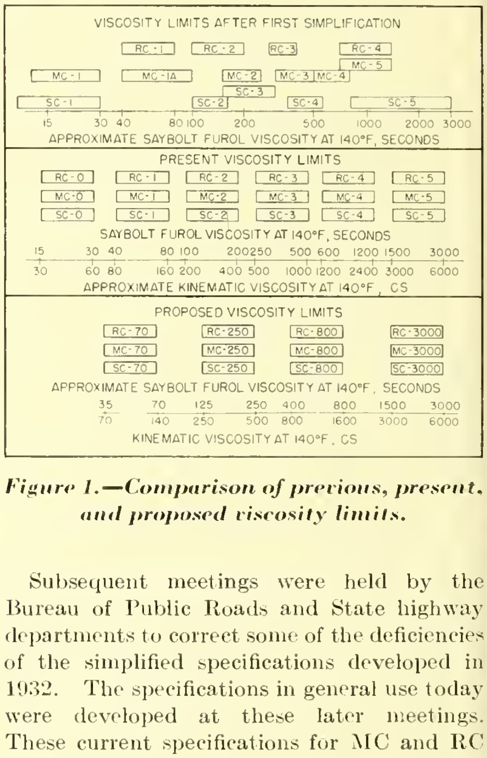

Figure 1.—Comparison of previous, present, and proposed viscosity limits.

Subsequent meetings were held by the Bureau of Public Roads and State highway departments to correct some of the deficiencies of the simplified specifications developed in 1932. The specifications in general use today were developed at these later meetings. These current specifications for MC and RC cutback were adopted by AASHO in 1942 and those for SC liquid asphalts were adopted in 1949. In these specifications, each type of liquid asphalt has been divided into six grades numbered 0 to 5, inclusive. The same viscosity range for a given grade has been established for SC, MC, and RC materials. The minimum and maximum limits of viscosity were selected so that each grade of material would be covered by approximately the same increment of viscosity at 140° F. when the viscosity was plotted on a logarithmic scale. The maximum viscosity permitted

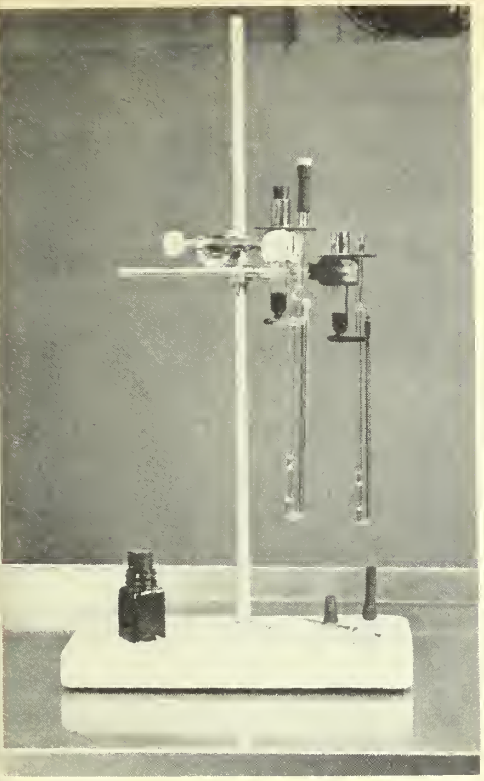

Figure 2.—Zeitfuchs cross-arm capillary viscometer.

for a given grade was specified at approximately twice the minimum viscosity permitted. Also, these revised specifications provided for a small but definite gap between the maximum viscosity permitted for one grade of material and the minimum viscosity established for the next heavier grade. The revised limits for viscosity are also shown in figure 1.

Although furol viscosity at 140° F. was used as the basis for establishing the limits of the various grades, actual tests could not be made at this temperature on all grades of material because of limitations of the furol instrument. For the most fluid grades of asphalt, flow from the furol viscometer is too fast at this temperature and inaccurate measurements are caused by turbulent flow; and for the heavy grades, flow is too slow at this temperature for accurate results. Thus, it was necessary to establish the approximate equivalents at other temperatures. Four temperatures varying from 77° F. to 180° F. were used to measure furol viscosity of the six standard grades of materials. A temperature of 210° F. has also been used for more viscous materials, such as the SC-6 grade specified by AASHO.

This limitation of instrumentation has now been overcome by the design of capillary tube instruments that measure the viscosity of opaque liquids, and by the availability of such viscometers in a range of capillary tube diameters. With these instruments the kinematic viscosity for all grades of liquid asphaltic materials can be determined rapidly, and with a high degree of precision, at 140° F.

Because a relatively precise conversion can be made from furol viscosity to kinematic viscosity, no fundamental problems are involved in changing the specification control of the liquid grades of asphaltic materials from furol seconds to the kinematic viscosity in centistokes. The kinematic viscosity in centistokes is very close to twice the furol viscosity, and one value can be easily converted to the other.

Several capillary tube instruments of various designs are available; all give satisfactory results, but the Zeitfuchs-type capillary appears to be the most popular. This tube is illustrated in figure 2. With this instrument the kinematic viscosity is determined by measuring the time of flow of the sample between the two calibration points on the tube. Viscosity is then a constant multiplied by the time. The constant for each tube is dependent on the diameter of the tube and volume between the calibration points, and is determined by calibration with a standard oil of known viscosity obtained from the National Bureau of Standards or the Cannon Instrument Co., State College, Pa.

An important step toward the practical application of kinematic viscosity to control the consistency of liquid asphalts was taken at the Fourth Pacific Coast Conference [2] on Asphalt Specifications held at San Francisco, Calif., May 10–11, 1961. This Conference adopted recommendations to: (1) Reduce the number of grades of each type of liquid asphalt from six to four, (2) realine the viscosity limits, and (3) express these limits in units of kinematic viscosity at 140° F. The Conference also recommended that the Saybolt furol method of test be retained as an alternate, with the results being converted to kinematic viscosity in centistokes. Figure 1 shows the viscosity limits for the four grades and a comparison of these limits with those of the present AASHO specifications. The grades are designated by type and the numerical lower limit of kinematic viscosity, such as SC–70, MC–70, RC–70, etc. These new specifications were effective January 1, 1962, in those States accepting the recommendations of the Conference.

The advantages to the materials and construction engineers resulting from adoption of these new specification controls can be briefly summarized, as follows: (1) All determinations made in the materials laboratory can be made from tests conducted at one temperature, 140° F., instead of the four or five temperatures now required; (2) experience has shown that time will be saved in testing; (3) tests can be made more economically because additional capillary viscosity tubes cost less than extra tubes for the furol viscometer; (4) better precision of test results can be expected; (5) the availability of results in fundamental units will permit a more scientific evaluation of the relation of consistency to the road behavior of these materials.

The Problems With Respect to Asphalt Cements

In the foregoing discussions, it was indicated that there were no serious problems and a number of advantages with respect to specifying kinematic limits for liquid asphaltic materials. However, the adoption of fundamental units to asphalt cements, with the complete elimination of the penetration test, presents very complex problems. These problems are: (1) Asphalt cements differ widely in viscosity-temperature susceptibility so that materials of equal viscosity at one temperature may have widely different viscosities at other temperatures. (2) Asphalt cements at atmospheric temperatures exhibit complex flow properties. The degree of complex flow differs for asphalts produced from different crude sources and by different methods of refining. (3) The degree of complex flow changes with temperature changes for individual asphalts; it also changes during hardening in service.

These same problems also apply to the application of absolute viscosity as a specification control for residues from the distillation of the cutback asphalts. The primary problems, nonuniformity and varying behavior in service of asphalt cements meeting the same specification, arise from the difference in viscosity-temperature susceptibility of different materials. It is well known that a group of asphalts, having identical penetrations and also identical viscosities at 77° F., can have widely different viscosities at high temperatures, such as those used for mixing. Viscosities at 140° F., the usual maximum for pavement temperatures, can also be significantly different. These differences are of considerable concern in a number of States, especially those in which a wide variety of types of asphalts are available. A number of suggestions have been made for minimizing the differences; such as placing minimum or minimum and maximum limits on the furol viscosity of the asphalt at 250° F. or 275° F. At present there also is considerable interest in the possibilities of basing the control of asphalt cement grade on the absolute viscosity at 140° F. Such control standards appear reasonable at first glance; but, a study of the possible variations under such conditions shows that specification control by viscosity at only one temperature, 140° F. may not be sufficient and could be even less desirable than the present standard for control at 77° F. by means of penetration.

Effects of change to control at 140° F.

Figure 3 was constructed to illustrate the possible effects of changing from control by penetration at 77° F. alone to viscosity measured at the single temperature of 140° F. The curves are approximations based on the assumption that the log of the log of viscosity in centipoises plotted against the log of absolute temperature in degrees Rankine (° F.+459.7) gives a straight line over the entire range of temperature from 20° to 350° F. Actual curves for asphalts generally show good

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[2] States included in the Pacific Coast Conference on Asphalt Specifications are: Arizona, California, Idaho, Montana, Nevada, Oregon, and Washington.

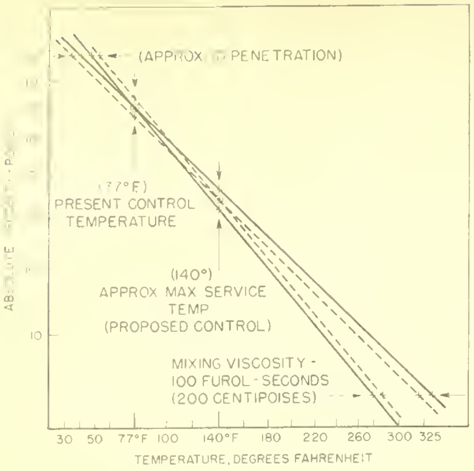

Figure 3.—Maximum and minimum viscosity-temperature relationships for 85–100 penetration grade asphalts.

conformity to this concept but there are indications that some materials deviate from a straight line because of unusually complex flow characteristics.

The slopes of the lines in figure 3 represent the extremes encountered with commercially available 85–100 penetration grade asphalts in the recent study made by the Bureau of Public Roads.[3] The solid lines are arbitrarily drawn to cross at 77° F. at a viscosity of 1 megapoise (one million poises), which is approximately equivalent to a penetration of 93. The figure 93 was calculated from a general relation between viscosity and penetration that was developed from actual tests made on a large number of asphalts in the Bureau of Public Roads laboratory. This relationship agrees closely with published relationships.

The dotted lines were obtained by bringing the curves together at 140° F. (the suggested point of control) in terms of viscosity. Table 1 shows the effect of maximum differences in viscosity-temperature susceptibility at several points of major interest. First, when control is based on penetration measured at 77° F., it is indicated that two materials having the same viscosity at 77° F. could have viscosities as low as 1,500 poises and as high as 4,000 poises at 140° F. The mixing temperature based on a viscosity of 100 furol seconds (200 centipoises or 2 poises) would be 273° F. for the material of high viscosity-temperature susceptibility, and 332° F. for the material with low viscosity-temperature susceptibility. There is no specific information that can be used to show precisely the point at which a pavement becomes brittle, but it is believed that a relative indication of this tendency can be obtained by considering the temperature at which the viscosity reaches a value of 100 megapoises, which is approximately equivalent to a penetration of 10. The range of temperature for the solid curves in figure 3 is from 41–48° F.

If the control is shifted to 140° F., as shown by the dotted lines, it becomes evident that differences in viscosity of the asphalt would be eliminated. Differences in temperature at the mixing viscosity would be reduced, the minimum temperature becoming 284° F. and the maximum 319° F. However, at a temperature of 77° F. or lower the possible variations encountered are much greater than might be expected. The curves show that the material most susceptible to temperature variations would have a viscosity of 2.3 megapoises, which is approximately equivalent to a penetration of 63; whereas, the material with the least susceptibility would have a viscosity of 0.5 megapoise, which is equivalent to a penetration of 132. The temperature range for 100 megapoises viscosity (approximately 10 penetration) would also increase; the new minimum would be 34° F. and the maximum 53° F.

The general curves in figure 3 are based on two materials having the same viscosity at the center of any given (or proposed) grade limits. However, actual variations could be considerably greater if the maximums and minimums of the asphalt grades were considered. These possibilities are shown by the values in table 1. It is of particular interest to note that, if viscosity limits at 140° F. of 1,500 to 3,000 poises were used as the limits for a particular grade, differences in penetration at 77° F. could range from a minimum of 48 to a maximum of 195. Although it may be possible to design pavements making use of materials with such different penetrations, it is obvious that some control is needed to prevent them from being supplied by the same specification.

Most asphalt technologists agree that it may be necessary to control consistency at two temperature points (or possibly three), thereby reducing the possibility of having materials with such differences meeting the requirements of the same specifications. At the present time, it is not possible to recommend what these points should be. However, from consideration of the various possibilities illustrated in figure 3 and table 1, it appears that one of the control points should be at 77° F. or lower.

Up to this point, only the problems related to the differences in viscosity-temperature susceptibility of the various asphalts have been discussed. It has been assumed that the deviations of paving asphalts from true liquids are not sufficiently different to affect specification control. This assumption is valid for values at 140° F. and above, but for values at lower temperatures, the complex flow characteristics of many materials would significantly affect the measured viscosity. The results of tests made at different shear rates give different apparent viscosities. To provide comparable data, it has become customary to report the apparent viscosity of all asphalt materials at a "standard" shear rate. Several rates or systems of reporting have been suggested for different viscosity ranges, but more information is needed as to which rate most nearly relates to the shearing forces in the pavement. Viscosity differences at different shear rates further emphasize the need to have at least one of the control points at a temperature appreciably below 140° F.

Instruments Available

It is important that instruments be available that will permit rapid and precise viscosity measurements in the laboratory. As mentioned earlier, the wide range of viscosity in asphaltic materials makes it impracticable to obtain results by using only one instrument. However, various instruments now available will permit coverage of most of the desired range.

One of the most popular viscometers used in asphalt research in the last few years has been the sliding plate microviscometer (fig. 4). This instrument, developed by the Shell Development Co., is useful for determining viscosities from approximately 100 thousand to 100 million poises. For asphalt cements, this represents a temperature range from a low of 40° to 50° F. up to a high of from 120° to 140° F., depending on the grade of material.

The test specimen for the sliding plate microviscometer is a "sandwich" of asphalt between two glass plates. In operation, one of the plates is held rigidly in the instrument and the other is attached to one part of a balance arm. A load is applied to the opposite end of the arm and the movement of the one plate with respect to the other is recorded. This provides data from which the rate of shear at various shearing stresses can be calculated. The apparent viscosity

Table 1.—Effect of differences in viscosity-temperature susceptibility for asphalts of same grade [1]

	Present control at 77° F.		Proposed control at 140° F.	
	Variation for equal values at center of grade	Total possible variation	Variation for equal values at center of grade	Total possible variation
Penetration at 77° F., 100 g., 5 seconds	[2] 93	[2] 85–100	63–132	48–195
Viscosity at 77° F. megapoises	[2] 1.0	0.88–1.25	0.50–2.30	0.22–4.0
Viscosity at 140° F. poises	1,500–4,000	1,300–5,500	[2] 2,300	[2] 1,500–3,000
Temperature range for 100 megapoise viscosity, (approximately 10 penetration) ° F	41–48	38–50	34–53	30–57
Temperature range for 200 centipoise viscosity, (approximately 100 furol-seconds) ° F	273–332	270–337	284–319	268–324

[1] Values shown are approximations based on general relationships and maximum and minimum susceptibility of asphalts produced in the United States.
[2] Control point.

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[3] *Properties of Highway Asphalts, Part 1, 85–100 Penetration*, by J. Y. Welborn and W. J. Halstead, PUBLIC ROADS, vol. 30, No. 9, Aug. 1959, pp. 197–207.

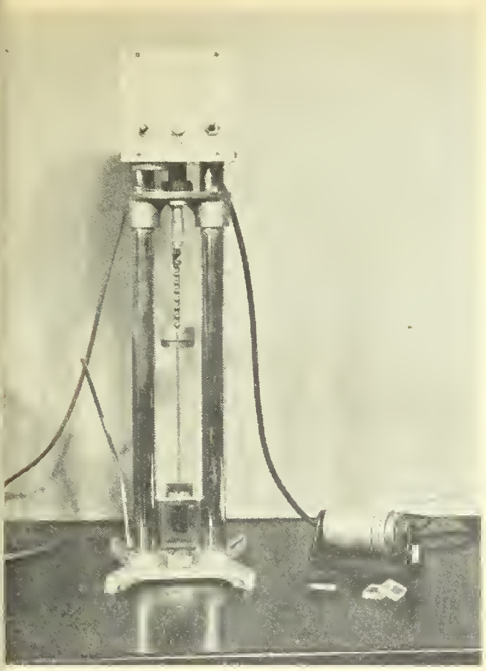

Figure 4.—Sliding plate viscometer.

at a predetermined "standard" shear rate is reported as the viscosity of the material. This instrument cannot be used to measure the viscosity of most asphalts at 140° F. because of the difficulty of holding the specimen in place and the extremely low shearing stresses involved. Several instruments, such as the rotating cylinder-type or the vacuum-operated capillary viscometer, are available that can be used to measure the viscosity of asphalts at this temperature.

At present, the greatest interest is being shown in the use of the vacuum-operated capillary-type of viscometer, which is illustrated in figure 5. With this device, the rate of rise of the liquid in a capillary tube is measured for specific values of vacuum applied at the top. The basic equation for capillary flow can then be used to calculate the absolute viscosity. Different sizes of capillaries, different heights of rise, and different amounts of vacuum can be employed to provide measurements over a relatively wide range of viscosities. Figure 5 shows the Koppers tube, which was developed some time ago; other designs of vacuum-operated capillary instruments, which employ the same principle of operation, also are now available.

A Zeitfuchs-type capillary instrument, previously discussed, is convenient to use for determining the viscosity of asphalt cements at high temperatures, such as those used for mixing. The furol instrument can also be used at these high temperatures and the results converted to kinematic units of viscosity.

With a combination use of the sliding plate, vacuum-operated capillary, and Zeitfuchs viscometers, the range of viscosities from 1 poise to 100 million poises can easily be covered, but these instruments cannot provide all the measurements considered desirable. Good instrumentation to measure viscosity at temperatures around 0° to 40° F., where viscosities are in the range of 1 billion to possibly 1,000 billion poises, is needed.

It is believed also that a concerted effort should be made to establish the relative importance of rheological differences in binders, such as shear susceptibility, age hardening, or gelling; these factors are not fully taken into account by the present practice of expressing viscosity at a "standard" shear rate. In particular, studies at low temperatures are needed to develop the relation of pavement brittleness to the properties of the binder. Instruments are also needed to measure apparent viscosities at such low temperatures. While the asphalt industry in general has shown, and continues to show, a very cooperative attitude with respect to such studies, the primary responsibility to establish overall relationships and to translate these relationships in terms of specification requirements rests with the various State highway departments. It is hoped that the States, through their own research and the cooperative research with the Bureau of Public Roads, will accept the challenge that these problems represent.

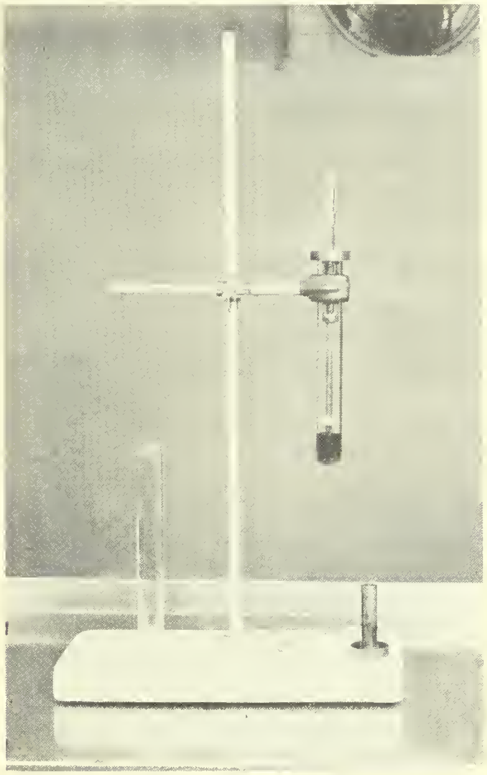

Figure 5.—Vacuum capillary viscometer.

Summary

To summarize, the problems relating to specification control of bituminous materials have been shown to be quite different with respect to the liquid asphaltic products and the asphalt cements. The present specifications for liquid asphaltic road materials are based on relationships of furol viscosity at 140° F. for all grades, and the changeover to specifications based on kinematic units at 140° F. could be accomplished easily because the furol viscosity can be converted to kinematic units with satisfactory accuracy. The primary advantages to such a changeover would be the convenience of being able to make all laboratory tests at the same temperature, and the resultant ready availability of the data in fundamental units, which would permit direct comparisons of different grades and types of liquid asphaltic materials.

It is very important that the rheological properties of asphalt cements be considered before viscosity limits based on temperatures other than those now used are established. It is recognized that the present empirical methods have shortcomings, and that studies should be continued toward the establishment of controls on the basis of viscosity tests. The use of fundamental consistency relationships should be introduced into pavement design and research studies. However, one-point control of viscosity, such as at 140° F., could possibly create greater differences with materials of the same grade than now exist with present grades and controls. Therefore, careful study of the effects of such a change and the possibility of establishing controls at two or more temperatures should be considered before viscosity units are adopted for grading asphalt cement.

Comparison of Zeitfuchs Cross-Arm and Saybolt Furol Viscometers for Measuring Viscosity of Cutback Asphalt

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported by EDWARD R. OGLIO and JOSEPH A. ZENEWITZ,
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Bituminous Materials and Chemical Section

The use of the Zeitfuchs cross-arm viscometer has recently been proposed for measuring the kinematic viscosity of cutback asphalt. This proposal is largely a result of the current trend toward measurement of the properties of bituminous materials by fundamental methods rather than by the customary empirical methods. The work reported in this article was undertaken to determine the feasibility of the instrument for this purpose.

A technique was developed for determining, with a high degree of precision, the kinematic viscosity of cutbacks with the Zeitfuchs viscometer. A comparison was made with the presently specified Saybolt furol viscometer. It was found that, with the new technique, significantly better precision was obtained with the Zeitfuchs viscometer than with the Saybolt furol instrument, and that the former was more convenient to use. The feasibility of using the proposed instrument for determining the viscosity of all grades of cutback at the same temperature was confirmed.

Conclusions

Results obtained from tests with both the Zeitfuchs cross-arm capillary viscometer and the Saybolt furol viscometer confirmed the feasibility of using the Zeitfuchs instrument for the determination of kinematic viscosity for all grades of cutback asphalt. This was specifically demonstrated by tests made at 140° F. with cutback asphalts of the present specification grade 5 and with the proposed specification grades of light (grade 70), light medium (grade 250), and heavy (grade 3,000) for both the rapid- and medium-curing types.

With the procedure developed during this study better precision for viscosity determinations of all grades of cutback asphalts may be expected with the Zeitfuchs viscometer than with the Saybolt furol viscometer. This procedure permits precise control of the volume of asphaltic material used in the test and the careful treatment of the asphalt sample, which are believed to be necessary for obtaining good repeatability of results.

Introduction

CONSIDERABLE interest exists in a move to establish absolute viscosity as the standard for the measurement of consistency of bituminous materials. (See the article, *Absolute Viscosity as a Specification Control for Bituminous Binders*, appearing elsewhere in this magazine.) This interest has been generated by two factors—the trend toward measurement of fundamental properties of materials for the control of quality in production and research, and the present availability of practical instruments with which to make such measurements.

One of the various proposals that has been made in this area is that the Zeitfuchs cross-arm capillary viscometer be used to determine the kinematic viscosity of cutback asphalts and that kinematic viscosity be used as the measurement of consistency rather than, or as an alternate to, the presently specified Saybolt furol viscosity (*1*).[1] The Zeitfuchs cross-arm instrument is suitable for measuring viscosity of opaque liquids and, because of its availability in a variety of capillary diameters, it is also useful for making viscosity determinations at one temperature for all grades of cutback asphalts.

Although some State highway departments have used the Zeitfuchs viscometer to determine viscosity for both asphalt cements and cutback asphalts, the producers of asphalt probably have used it to a greater extent. The Phillips Petroleum Co. reports that it has used this instrument for a number of years to control the viscosity of its cutback products. This company has urged that kinematic viscosity be included as an alternate to the Saybolt furol viscosity requirement in current specifications for cutback asphalts (*2*, *3*).

The comparison of the two viscometers reported in this article was made in an effort (1) to develop a technique for utilizing the Zeitfuchs cross-arm viscometer that might be suitable for inclusion in specifications, and (2) to obtain data for estimates of precision that can be expected when this instrument is used to measure viscosities of cutback asphaltic materials.

Laboratory Tests

Three replicated determinations of viscosity on each of four grades of rapid curing (RC) and medium curing (MC) types of cutback asphalt were made in the laboratory with both the Zeitfuchs and Saybolt furol viscometers. The determinations with the Zeitfuchs tubes were made at a temperature of 140° F., in accordance with the ASTM method (*4*). The Saybolt furol determinations also were made in accordance with the applicable ASTM method (*5*), at temperatures appropriate for each grade. The samples were not filtered as specified in these test designations because they contained appreciable amounts of volatile thinner. The three determinations representing a replicated set were made simultaneously for the Saybolt furol viscosity tests and consecutively for the kinematic viscosity tests.

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[1] References indicated by italic numbers in parentheses are listed on page 251.

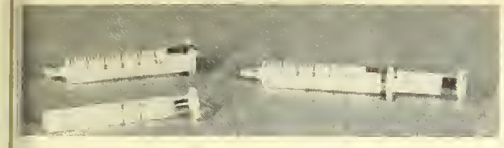

Figure 1.—Luer hypodermic syringe used to fill viscometer.

The test data were treated by the usual statistical procedures to determine the precision of results obtained for each instrument for each type of cutback.

Materials used in tests

The basic materials used in the preparation of the cutbacks for the comparison tests are described in the following list.

Grade 5 rapid curing cutback asphalt, with Saybolt furol viscosity of 327.6 seconds at 180° F.

Rapid curing cutback thinner (cutter stock).

Grade 5 medium curing cutback asphalt, with Saybolt furol viscosity of 408.6 seconds at 180° F.

Medium curing cutback thinner (cutter stock).

The base cutback asphalts were thinned with the proper volumes of the appropriate diluent to obtain materials within the viscosity ranges for the heavy (grade 3,000), light medium (grade 250), and the light (grade 70) grades of cutback materials proposed at the Fourth Pacific Coast Conference on Asphalt Specifications, May 1961. The Saybolt furol viscosities for these grades were approximately 300 seconds at 180° F., 200 seconds at 140° F., and 75 seconds at 122° F., respectively. All materials were stored in friction top cans and the determinations were made within one week after formulation of the test material.

Detailed testing procedure

The formulated test material was thoroughly stirred while at room temperature; care was taken to avoid incorporation of air. For the Saybolt furol tests, this thoroughly stirred material was poured into three furol tubes; for the kinematic viscosity tests with the Zeitfuchs viscometer, the material was poured into six 3-ounce, rubber-stoppered bottles. Six specimens were taken for the kinematic viscosity determination so as to permit a comparison for each material of the results obtained with Zeitfuchs viscometer tubes of two different viscosity ranges.

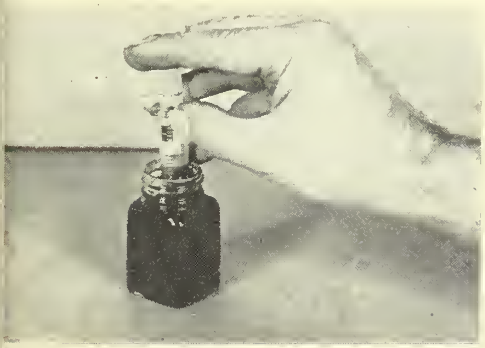

Figure 2.—Container for viscosity sample and illustration of technique for filling syringe.

In the Saybolt furol determinations, the viscometer bath and tubes had temperatures of from 0.5° to 1° F. above the test temperature when the samples were introduced into the tube. During the conditioning of the samples to test temperature, the tubes were kept covered and the contents were stirred continuously with the viscosity thermometers. From 20 to 25 minutes were required to obtain the test temperature. The three determinations constituting a replicated set were made simultaneously.

In the determination of kinematic viscosity with the Zeitfuchs tubes, a procedure was developed for the close control of the volume of the sample used in each replicated set of determinations. The necessity for controlling the volume of the sample had been indicated earlier, when the tubes were calibrated with the National Bureau of Standards (N.B.S.) standard viscosity oils to obtain the instrumental constants. During this calibration, in most cases, the filling line proved to be useless as a guide for introduction of the samples. With a number of tubes, the sample would spontaneously flow through the siphon and into the capillary. In addition, with the more viscous materials, such as the N.B.S. standard viscosity oils M, N, and P, it was practically impossible, by using the reference fill-line on the instrument, to control the volume of the sample for a determination. To accomplish this control and avoid having interfering droplets on the walls of the filling tube, a graduated Luer syringe was used to introduce the sample into the tube for tests made during this study.

The Luer syringe and its use in the filling of the viscometer are illustrated in figures 1–4. Because of photographic difficulties, only one hand is shown in these illustrations but two hands are required for some of the actual manipulations. The syringe, assembled and disassembled, is shown in figure 1. Special calibration markings were applied to the plunger. These calibration marks on the syringe's plunger were used in controlling the volume of the asphaltic material: the standard markings on the syringe are difficult to use with opaque materials. The type of container, and its relative size, used to hold the asphalt samples during conditioning and storage is shown in figure 2. The technique used to remove the air bubbles and adjust the volume of the sample in the syringe is illustrated in figure 3. In this procedure, material slightly in excess of that needed for the determination was drawn into the syringe, and the volume adjusted until the proper amount had been obtained; the extruded excess was removed from the syringe with paper toweling or cloth.

The viscometer and the syringe in place and ready for the filling operation are shown in figure 4. The test sample was discharged from the syringe at a slow steady rate by depressing the plunger with the forefinger. In this way, the rate of discharge easily was controlled to effect a quantitative transfer of the sample to the viscometer's reservoir without entrapment of air bubbles, which could have affected the results. Use of this technique also helped prevent the sample from touching or clinging to the upper walls of the reservoir and thus minimized the possibility of errors in results occurring from this source.

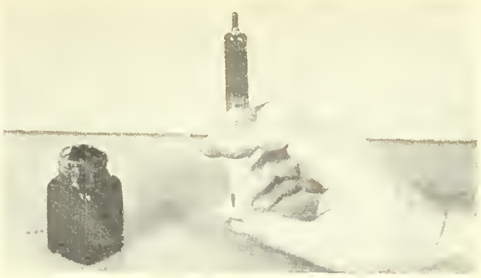

Figure 3.—Technique for adjustment of volume and elimination of air bubbles.

Efflux Time

Acceptable precision in efflux time was obtained with a material sample of 2.0 milliliters, for tubes applicable for viscosities of a range up to 3×10^3 centistokes; and with material samples of 3.0 milliliters, for tubes applicable for viscosities above 3×10^3 centistokes. These volumes, which corresponded to the volume of oil used in the calibration of the particular tube, were used for the kinematic viscosity determinations made during this study. The effect of the sample volume on efflux time obtained with the calibrating oils is shown by the figures in table 1.

Since a sufficient supply of Zeitfuchs viscometer tubes of any one particular range was not available, the replicated kinematic viscosity tests were made consecutively rather than concurrently as for the Saybolt furol determinations. The six 3-ounce, stoppered sample containers were immersed at staggered time intervals in a constant temperature bath held at 140° F., and allowed to condition for

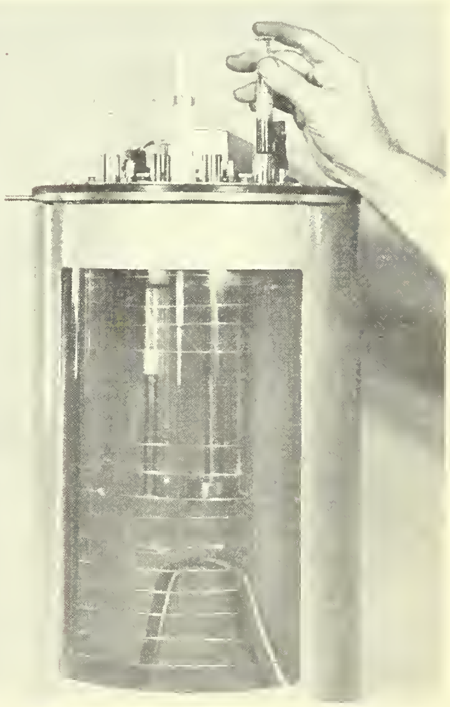

Figure 4.—Viscometer being filled from Luer syringe.

Table 1.—Effect of sample volume on efflux time with Zeitfuchs viscometers

Oil [1]	Temperature	Certification viscosity	Viscometer series	Volume of sample	Efflux time
	° F.	Centistokes	Number	Milliliters	Seconds
M	77	283.2	0.3	1.6 1.8 2.0	724.2 713.0 707.9
P	122	9,947.0	30.0	2.0 3.0	338.1 284.8

[1] N.B.S. standard viscosity oils, M and P.

Table 2.—Viscosity determinations and statistical summary

	Saybolt furol viscosity, seconds at—			Kinematic viscosity, at 140 ° F.—			
	122° F.	140° F.	180° F.	Viscometer[1] series	Centistokes	Viscometer[2] series	Centistokes
RAPID CURING CUTBACKS:				Number		Number	
RC 5 grade			327.6	30.0	3,620.0 3,616.4 3,616.4	10.0	3,584.5 3,604.5 3,611.1
Average					3,617.6		3,600.0
Standard deviation					2.1		13.9
Coefficient of variation percent					0.06		0.38
Light (grade 70)	83.7 84.5 83.6			1.0	110.25 110.06 111.15	0.3	109.91 110.07 111.54
Average	83.93				110.49		110.50
Standard deviation	0.49				0.58		0.90
Coefficient of variation percent	0.59				0.53		0.81
Light medium (grade 250)		192.5 201.0 199.0		3.0	408.75 412.30 410.36	1.0	407.73 407.91 409.00
Average		197.50			410.47		408.21
Standard deviation		4.44			1.78		0.69
Coefficient of variation percent		2.25			0.43		0.17
Heavy (grade 3,000)			305.2 308.5 315.4	30.0	3,181.1 3,170.5 3,230.2	10.0	3,189.7 3,209.7 3,221.9
Average			309.70		3,193.9		3,207.1
Standard deviation			5.20		31.9		16.3
Coefficient of variation percent			1.68		1.00		0.51
Pooled data coefficient of variation percent		2.01			0.69		
MEDIUM CURING CUTBACKS:							
MC 5 grade			408.6	30.0	4,448.6 4,459.1 4,462.6	10.0	4,449.6 4,449.6 4,418.7
Average					4,456.8		4,439.3
Standard deviation					7.3		17.8
Coefficient of variation percent					0.16		0.40
Light (grade 70)	86.0 87.0 85.0			1.0	110.97 110.61 111.33	0.3	110.91 111.35 110.91
Average	86.00				110.97		111.06
Standard deviation	1.00				0.36		0.25
Coefficient of variation percent	1.16				0.32		0.23
Light medium (grade 250)		212.4 213.6 209.6		3.0	444.29 444.94 445.26	1.0	443.15 444.23 443.67
Average		211.87			444.83		443.69
Standard deviation		2.05			0.49		0.54
Coefficient of variation percent		0.97			0.11		0.12
Heavy (grade 3,000)			290.0 287.0 291.6	30.0	2,956.4 2,991.5 2,956.4	10.0	2,947.9 2,977.9 2,974.6
Average			289.53		2,968.1		2,966.8
Standard deviation			2.34		20.3		16.5
Coefficient of variation percent			0.81		0.68		0.56
Pooled data coefficient of variation percent		0.96			0.58		
RC and MC pooled data coefficient of variation percent		1.58			0.63		

[1] Efflux time: Less than 200 seconds in each case.
[2] Efflux time: 200 seconds or more in each case.

20 to 25 minutes. After the conditioning period, the container was removed from the bath and unstoppered; the material was carefully stirred and about 0.5 milliliter more than the required volume was removed. This sample was taken by the Luer syringe which had been preheated to about 125° F. from a level about 1 centimeter below the surface of the liquid. Any excess material or air, which might have been incorporated by the withdrawal procedure, was then expelled as previously described and the sample introduced into the reservoir of the viscometer and conditioned for 7 minutes in the constant temperature bath, as specified by the ASTM method (*4*).

Both the reservoir tube and the outlet tube of the viscometer were kept stoppered during the 7-minute conditioning period, to minimize evaporation and to prevent premature siphoning. The procedure from this point was in accordance with the ASTM test method (*4*). Individual stoppered containers were used to furnish material for each replicated determination, because an appreciable increase in viscosity occurred with each opening of the container for sample withdrawal. It was noted that prewarming was unnecessary when sampling the light grade material.

Table 2 contains the results for the replicated viscosity determinations, which were obtained with both the Saybolt furol and Zeitfuchs viscometers, for the various laboratory prepared grades of asphalt cutbacks and the current grade 5 cutbacks (MC and RC).

Statistical Treatment

The average and standard deviation were computed for each set of replicated determinations. For standard deviation, the computation was made in accordance with the recommendations for small sets of measurements found in most texts on statistical methods, as shown by the following formula:

$$S.D.=\sqrt{\frac{\Sigma(X-\overline{X})^2}{n-1}}$$

where,

X = an individual observation (a determination in the set).

$\overline{X}$ = the average for a set of three replicated determinations.

n = number of observations (3 for each set reported in this article).

In order to compare the precision of the results obtained with the Zeitfuchs instrument with those obtained with the Saybolt furol instrument, the coefficient of variation was also computed for each set. This is the ratio of standard deviation to the average expressed as a percentage:

$$\text{Coefficient of variation}=\frac{S.\ D.}{\overline{X}}\times 100$$

Standard deviation and the corresponding coefficient of variation were computed (table 2) for each type material and for each instrument. This was done in accordance with the following equations:

Pooled S. D.=

$$\sqrt{\frac{\Sigma(X_a-\overline{X}_a)^2+\quad\Sigma(X_k-\overline{X}_k)^2}{(n_a-1)+\ \_\_\_(n_k-1)}}$$

Coefficient of variation=

$$\frac{\text{S. D. (pooled results)}}{\overline{\overline{X}}}\times 100$$

where,

$a\_\_k$ = the replicated sets

$(n_a-1)+\_\_\_\_(n_k-1)$ = sum of all the observations less one (for each set).

$\overline{\overline{X}}$ = Overall average for the group of replicated sets considered (average of the averages for the group).

Summary

A comparison of the coefficients of variation of the pooled results for all the cutbacks, given in table 2, shows that the Zeitfuchs viscometer provided better precision than the Saybolt furol instrument. It is of interest to note that with the Zeitfuchs tubes no significant difference in precision was obtained between the types of cutbacks (RC or MC). With the Saybolt furol viscometer, appreciably better precision was obtained with the medium curing cutbacks than with the rapid curing type (0.96 percent vs. 2.01 percent). This difference is attributed to the lower volatility of the diluent in the medium curing materials, which resulted in smaller losses of these volatiles during sample preparation.

However, the precision of results obtained with both instruments for each type of the tested cutbacks was estimated to be less than that stated in the ASTM methods (*4, 5*). This deficiency in precision is attributed to the volatile diluents in the cutback materials. This is confirmed by a consideration of the calibration data obtained with the three N.B.S. standard viscosity oils (table 3). With these oils, which contained no volatile diluents, the average deviation from the mean values of the efflux times was generally within the 0.1 percent level (*4, 6*). Furthermore, the coefficients of variation computed for the replicated sets, as well as for the pooled data, were significantly lower for these oils than for the cutbacks. For the pooled data these coefficients were 0.13 percent for the oils (table 3) and 0.63 percent for the cutbacks (table 2).

Comparison of convenience

The Zeitfuchs tubes were somewhat easier to use and were more conveniently cleaned than the Saybolt furol tubes. The time required to complete a test was about the same for each instrument, 20 to 30 minutes. The conditioning time required for tests made with the Zeitfuchs viscometer was only 5 to 7 minutes, as compared to the 20- to 30-minute conditioning time required for the Saybolt furol tests. However, additional time was required for the preparation of material for the Zeitfuchs viscometer tests to obtain samples sufficiently fluid. For routine work with larger volumes of material, this additional preparation time could be considerably reduced by warming the sample on a hot plate. For kinematic viscosity determinations for the light grades of material (present grades 0–2) this prewarming step could probably be eliminated.

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Table 3.—Zeitfuchs precision data from N.B.S. standard viscosity oils

Oil	Temperature	Certified viscosity	Viscometer series	Volume of oil	Efflux time	Average [1] efflux time	Deviation from average	S.D.[1]	Coefficient [1] of variation
Type	*° F.*	*Centistokes*	*Number*	*Milliliters*	*Seconds*	*Seconds*	*Percent*	*Seconds*	*Percent*
M	77.0	283.2	0.3	2.0	708.05 708.8 707.55	708.13	0.01 0.09 0.08	0.603	0.08
M	77.0	283.2	1.0	2.0	156.8 156.6 156.65	156.68	0.08 0.05 0.02	0.104	.07
N	77.0	1,033.0	3.0	2.0	319.8 319.4 320.1 319.4	319.73	0.03 0.09 0.13 0.09	0.132	.04
N	68.0	1,515.0	10.0	3.0	135.75 135.15 136.1	136.00	0.18 0.11 0.07	0.219	.16
N	68.0	1,515.0	10.0	3.0	137.2 136.5 136.2	136.6	0.44 0.07 0.29	0.515	.38
P	122.0	9,947.0	30.0	3.0	283.6 283.1 283.1 283.4	283.3	0.11 0.07 0.07 0.04	0.245	.09

[1] Pooled data showed: Average efflux time of 290.07 seconds, standard deviation of 0.375 seconds, and coefficient of variation of 0.13 percent.

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